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Waterways Experiment Station

Evaluation of Stone Matrix Asphalt

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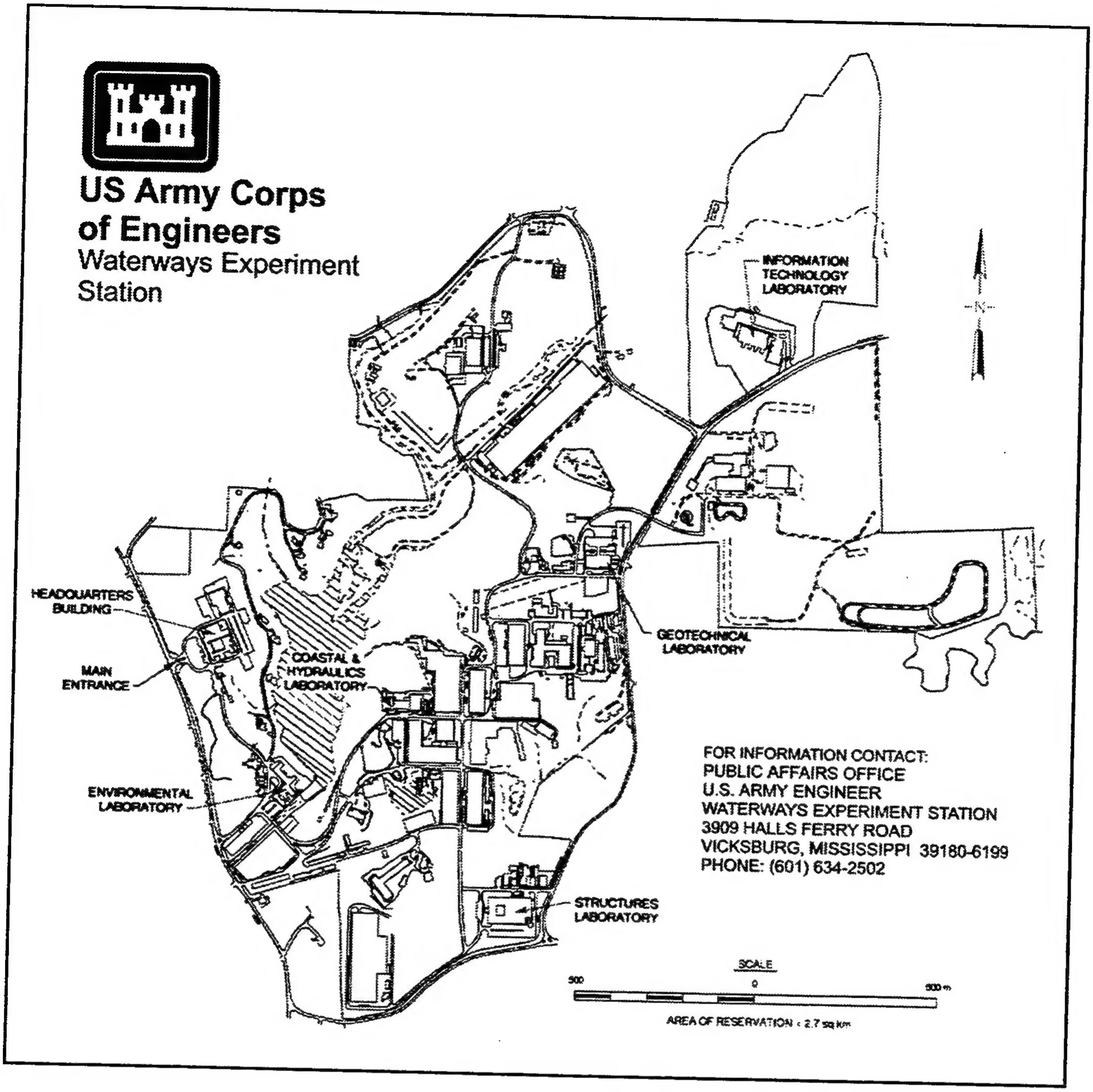

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Contents

Preface

This report describes the results of a series of laboratory and field studies conducted by the Geotechnical Laboratory (GL), U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, for the Air Force Civil Engineering Support Agency (AFCESA). The studies were conducted from October 1991 to December 1995. At the request of the sponsor, the report is being published under a WES cover as a technical report.

The study was conducted under the general supervision of Dr. W. F. Marcuson III, Director, GL; Dr. Raymond Rollings, Acting Chief, Airfield and Pavements Division (APD); Mr. Robert D. Bennett, Acting Chief, Soil and Rock Mechanics Division (SRMD); and Mr. T. W. Vollor, Chief, Materials Analysis Branch (MAB). The project's Principal Investigator was Mr. James E. Shoenberger, MAB. The report was written by Messrs. Shoenberger, and Lenford N. Godwin, Dr. Larry N. Lynch, MAB, and Dr. Paul A. Gilbert, Soils Research Center. The Air Force Technical Monitor was Mr. Jim Greene, AFCESA.

Director of WES during the conduct of this study and preparation of the report was Dr. Robert W. Whalin.

Conversion Factors, SI to Non-SI Units of Measurement

SI units of measurement used in this report can be converted to Non-SI units as follows:

Multiply	By	To Obtain
centimeters	0.3937	inches
kilograms	0.001102	tons (2,000 pounds, mass)
kilograms per cubic meter	0.062422	pounds per cubic foot
kilograms per square meter	1.843345	pounds (mass) per square yard
kilopascals	0.145038	pounds (force) per square inch
megagrams	1.102311	tons (2,000 pounds, mass)
megapascals	145.038	pounds (force) per square inch
meters	3.2808	feet
millimeters	0.0394	inches
Newtons	0.224809	pounds (force)
pascal-second	10	poises
square meters per second	100	centistokes

1 Introduction

Background

In 1990, a study team of pavement specialists from the United States toured several European countries to evaluate and review pavement materials, practices, and performance. This study team was known as the European Asphalt Study Tour (EAST) and consisted of members from the American Association of State Highway and Transportation Officials (AASHTO), Federal Highway Administration (FHWA), National Asphalt Pavement Association (NAPA), Strategic Highway Research Program (SHRP), Asphalt Institute (AI), and Transportation Research Board (TRB). One paving product that received a substantial amount of attention from the EAST group was stone matrix asphalt (SMA) (AASHTO 1991). SMA mixtures have been used in Europe for over twenty years and have shown a high resistance to plastic deformation (rutting) even when subjected to heavy traffic (ScanRoad 1991; Brown and Manglorkar 1993). The SMA mixture, which consists of a coarse aggregate skeleton component and an asphalt mastic component, is used as a surface course on heavy duty pavements. Coarse aggregates form the skeleton that provides aggregate to aggregate contact to support the loading and resist rutting. The mastic component, which fills the voids in the skeleton and binds the aggregates together, is formed by a mixture of asphalt cement, fine aggregate, mineral filler, and stabilizing agent (Haddock et al. 1993). SMA is rich in binder and low in void content. SMA evolved from Gussasphalt mixtures which are basically mastic asphalt concrete wearing surfaces (AASHTO 1991; van der Heide 1992). Gussasphalt mixtures have a high degree of durability because of high asphalt contents and almost no voids in the mixture. Although constructed with relatively hard asphalt cements, they have shown a tendency toward plastic deformation under load. SMA is an effort to counteract this by providing an aggregate matrix for strength and load carrying capability, while still providing durability through high asphalt cement contents (van der Heide 1992).

Many pavements are experiencing increases in traffic loads and tire pressures, which in turn have increased permanent deformation, or rutting (Brown and Cross 1992). These increases in loading and tire pressures are also being experienced in airfield pavements (Geller and Murfee 1994). One method to improve a dense graded hot asphalt mixture's (HMA) resistance to rutting is to compact the HMA to a higher density. However, with the higher density,

there must be a reduction in asphalt content in order to maintain the HMA voids within the specified range. As the asphalt content is reduced, and the density increased, the HMA becomes stiffer to resist the rutting (Regan 1987), but at the same time the durability of the HMA is reduced. The potential for cracking and raveling distress increase with this reduction in asphalt content. SMA offers the potential of reduced rutting through its aggregate skeleton and improved durability from the higher asphalt content without any increase in compaction effort.

SMA pavements were first constructed in the U.S. in 1991. They have been placed successfully by several state Departments of Transportation (DOT), usually on high volume roadways (Brown 1992a; GDOT 1992; Haddock, Liljedahl, and Kriech 1993). The SMA mixtures produced contained a wide range of materials and additives. Even though some problems were encountered during mixing and placement, satisfactory SMA pavements were built (Scherocman 1992).

Objective

The three objectives of this study were to: evaluate the potential of SMA for use in military pavements, to evaluate SMA properties as compared to standard airfield asphalt concrete mixtures, and to identify potential SMA problem areas.

Scope

The objectives were accomplished through a combination of literature reviews, discussions with pavement engineers, laboratory testing and evaluation, and field demonstrations. The laboratory testing included the use of the Marshall hand hammer and the Gyratory Test Machine to evaluate compaction and standard Marshall properties. Indirect tensile tests were run on both the standard and SMA mixtures for a comparison of the strength properties between the mixes. In addition, creep-rebound testing was used to evaluate the rutting potential of SMA mixtures. Two SMA field demonstrations projects were constructed to identify problems and to provide future performance information. The demonstration sites were constructed at Edwards AFB, CA and RAF Lakenheath, United Kingdom.

2 SMA Mixture

Material Composition

The basic concept of an SMA is that the coarse aggregates interlock to form a stone skeleton and the void spaces between these aggregate particles are filled with an asphalt mastic. The stone to stone contact of the coarse aggregates provide a high resistance to abrasion and permanent deformation, such as rutting. The asphalt mastic holds the aggregate particles together and provides durability to the SMA mixture (Kennepohl and Davidson 1992, Little, Dutt, and Syed 1991, and van der Heide 1992).

Aggregates

As shown in Figure 1, there are significant differences between a typical airfield dense gradation and an SMA gradation. The SMA aggregate blend is more gap-graded than the dense graded blend for the coarse aggregate or material retained on the 4.75 mm (No. 4) sieve. In addition, the amount of material passing the 75 μm (No. 200) sieve is higher in the SMA than the dense aggregate blend. Figures 2 and 3 are schematic representations of SMA and dense graded mixtures. The coarse aggregates in the dense-graded mixture tend to float in the finer aggregates and asphalt binder, but the coarse aggregates in the SMA exhibit stone to stone contact. Since SMA mixtures depend on aggregate to aggregate contact to support traffic loads without rutting, it is essential that the aggregate blend is high in coarse aggregate. Too much fine aggregate or to much mastic will force the coarse aggregate particles apart destroying the stone skeleton or matrix needed to resist rutting (Haddock et al. 1993 and AASHTO 1991).

Even though not all European countries use the same SMA gradations, distinctive gradation parameters have emerged (Table 1). These parameters suggest that the coarse aggregate, which is aggregate retained on the 2.36 mm (No. 8) (2 mm, European standard size) sieve, make up 70 to 80 percent (20 to 30 percent passing) of the weight of the aggregate. The fine fraction or sand size fraction, defined as the material passing the 2.36 mm (No. 8) sieve but retained on a 1.18 mm (No. 16) (0.09 mm, European standard size) sieve, should comprise 12 to 17 percent of the aggregate weight.

One study (Brown 1992b) investigated the effects on various SMA mixture properties with variations in the amount of fine material in the mixture, see Table 2. This study found that the material passing the 1.18 mm (No. 16) sieve, should account for 8 to 13 percent of the aggregate weight (AASHTO 1991). Some European users and producers of SMA restrict the amount of aggregates passing the 2.36 mm (No. 8) sieve to 20 to 23 percent (77 to 80 percent retained) but suggest the amount be maintained closer to the 20 percent limit in order to maintain the stone matrix (AASHTO 1991 and Bukowski 1991). Figure 4 illustrates the relationship between the percent of coarse aggregate to the percent voids in the aggregate gradation. An increase in coarse aggregate from 40 to 70 percent has a minimal effect on the percent voids, but an increase in coarse aggregate from 75 to 80 percent produces a dramatic increase in the amount of voids (van der Heide 1992). Figure 4 reinforces the suggestion that a high amount of coarse aggregate, in the order of 80 percent, is needed to maintain an adequate void structure in SMA. In keeping with the concept of high coarse aggregate content, a technical working group sponsored by the FHWA developed a set of model guidelines for SMA. These guideline recommended the SMA gradation provided in Table 3 and shown in Figure 5 (NAPA 1994).

In addition to a desirable high percentage of coarse aggregate, a high degree of internal friction is crucial to prevent rutting. The aggregate particle shape and surface texture contribute to the internal friction. Therefore, a crushed aggregate is desirable. The coarse aggregate fraction typically is 100 percent crushed stone. The fines fraction, sometimes referred to as the sand fraction, is also usually 100 percent crushed aggregate (AASHTO 1991). The use of natural sands is not encouraged. Other aggregate qualities that are desirable include low Los Angeles abrasion values and aggregates that are low in percent flat and elongated particles (Scherocman 1992). Table 4 provides guidance on recommended basic aggregate property limitations (NAPA 1994).

Mastic

The voids in the SMA stone matrix or skeleton are filled with an asphalt mastic. This mastic is composed of asphalt cement binder, crushed fine aggregate, mineral filler, and a stabilizer. It is important that the amount of mastic be controlled to provide sufficient material to hold the aggregate particles together without being excessive to where the coarse aggregate matrix point to point contact is diminished (Carsick et al. 1991, NCAT 1991, and AASHTO 1991). The mastic forms a thick film on the coarse aggregates and fills much of the void space within the aggregate skeleton. This thick film with its high asphalt content is conducive to excellent low temperature properties and reduced age hardening, which in turn can result in more durable pavement (ScanRoad 1991).

Asphalt cement. In Europe, SMA mixtures utilize asphalt cement contents that typically exceed 6 percent, by total weight of mix. This high asphalt content adds to voids total mix (VTM) for SMA of approximately 3 percent

(Brown and Manglockar 1993 and Bukowski 1991). The potential for improved durability is also enhanced by the use of a high asphalt content (NAPA 1994). Asphalt cements with penetrations ranging from 60 to 80 are typically used in Europe. These penetrations are approximately equivalent to AC-20 viscosity graded asphalt (Bukowski 1991). Experimental SMA test sections that have been placed in the United States have included AC-20, AC-30, 85-100 pen, and multi grade 20-40 asphalt cements as binders (Brown 1992).

Mineral filler. The term mineral filler refers to the portion of the aggregate that passes the 75 μm (No. 200) sieve. These include limestone dust, hydrated lime, portland cement, and fly ash. As presented in Table 1, the amount of aggregate filler (dust) passing the 75 μm (No. 200) sieve is typically 8 to 13 percent for SMA mixtures used in Germany (Stuart 1992). The NAPA (1994), in its recommended guidelines, suggests that the filler content range from 8 to 10 percent. In both cases, the amount of filler in SMA is much higher than usually found in dense graded HMA, shown in Figure 1. The amount of filler is important in terms of obtaining the desired mixture air voids and in effecting the optimum asphalt content (Brown and Manglorkar 1993). Since the asphalt content is sensitive to the aggregate fines and filler content, the gradation of the filler can have an effect on the asphalt content of the SMA (Kennepohl and Davidson 1992). The normal film thickness of asphalt cement on the aggregate in an SMA is approximately 15 microns (0.00059 in.); therefore, any filler in this size range can act as an extender and reduce the amount of asphalt cement required. Filler that is relatively coarser will require additional asphalt cement to coat this surface area and will increase the amount of asphalt cement required. In Europe, SMA mixtures commonly employ a filler-to-asphalt ratio of approximately 1.5. In contrast, conventional dense graded HMA in the United States typically recommend a filler-to-asphalt ratio of less than 1.2 (Bukowski 1991 and AASHTO 1991). A filler with a uniform size distribution as possible and with no more than 20 percent of the total filler smaller than 0.2 mm is desired in the European SMA production. The common use of fillers not meeting this requirement is probably the reason that U.S. SMA mixtures tend to have lower asphalt contents than European SMA mixtures. In addition, in Europe, most filler is added by mineral filler feeder systems because baghouse fines are not normally fed back into the SMA (Scherocman 1992 and Stuart 1992).

Stabilizers. Because of the high asphalt content in the mix, the thick asphalt coating on the coarse aggregate, and the high voids in the aggregate skeleton, there is a tendency for the asphalt binder to drain from the aggregate during storage, transportation, or placement. In order to reduce this drainage potential, stabilizers are used to stiffen the mastic or to increase the asphalt binder viscosity. Originally, asbestos fibers were used as the stabilizer to stiffen the mastic (ScanRoad 1991 and van der Heide 1992). Today the most common materials used to stabilize the mastic include cellulose fibers or mineral fibers. Typical dosage rates for cellulose fiber is 0.3 percent by weight of mix while the rate for mineral fiber is approximately 0.4 percent by weight of mix (Scherocman 1992). Fiber length varies from about 0.64 cm (0.25 in.) to less than 0.25 cm (0.1 in.). Cellulose fibers are supplied as loose material

(packaged in premeasured units) for batch mixing plants and as pellets (50 percent fiber and 50 percent asphalt cement) for drum mixing plants. The use of fibers can reduce asphalt drainage by 10 times the amount that occurs in SMA mixtures without fiber stabilization (Little, Dutt, and Syed 1991). NAPA (1994) provides guidelines for SMA fiber stabilizers. These fiber guidelines are summarized in Tables 5 and 6. Polymer modifiers can also be used to increase the asphalt viscosity and reduce asphalt drainage. In addition, polymer additives are used to improve the low temperature properties of the binder material (Brown and Manglorkar 1993). Polymer modifiers are sometimes preblended with the asphalt cement and then are added to the SMA aggregates in the normal wet mixing phase. Other polymers are mixed with the hot aggregates in the dry mixing cycle before the asphalt cement is added to the aggregates. The amounts of polymer typically used varies from 3 to 8 percent by weight of asphalt cement (Scherocman 1992). It was reported by Bukowski (1991) that nearly 95 percent of the SMA produced in Germany utilizes cellulose fiber and that the use of polymers in lieu of fibers is rare. The Georgia Department of Transportation (GDOT) has incorporated both fiber and polymer in the same SMA mixture in some of their highway test sections (GDOT 1992). They have continued to use polymer with most of their SMA construction.

The effectiveness of the stabilizer is often evaluated by a drainage test. A German test, referred to as the Schellenberger test, and the FHWA 2.36-mm sieve test both measure the SMA weight loss during a hot storage cycle (Stuart and Malmquist 1994). A loss of less than 0.2 percent in the weight of the SMA is satisfactory, but a loss in excess of 0.3 percent indicates that a drainage problem may exist (Mogawer and Stuart 1994 and NAPA 1994). Similar work on drainage tests has been conducted by the National Center for Asphalt Technology (Brown and Mallick 1995). A National Asphalt Pavement Association publication (NAPA 1994) provides a suitable draindown procedure.

Mixture Design

In Europe, the SMA mix design procedures vary in minor details from one country to another. Much of the details incorporate each country's own experience, but the basic procedures used are the Marshall procedures. The 50 blow Marshall compaction is the common compactive effort used in Europe and has been the standard used on all the SMA test sections placed in the United States (AASHTO 1991 and Brown 1992). If the compactive effort is increased to more than the 50 blows per side, there is concern in Europe that aggregates will be fractured without any increase in density; therefore, increasing the SMA compactive effort is not encouraged (Stuart 1992). Along with the Marshall compaction, a target value for voids total mixture (VTM) is also used to help select the optimum asphalt content, usually 3 percent. Sometimes a minimum asphalt content is specified in conjunction with the target VTM. Meeting the minimum asphalt content and the target VTM may require some adjustment in the aggregate gradation. The most common adjustment is a reduction in the

amount of material passing the 4.75 μm (No. 4) sieve, resulting in an increase in the voids in the mixture.

The selection of an optimum asphalt content is sometimes based on Marshall stability and flow values. However, because of the percentage of coarse aggregate, the stability curve may not break over or reach a well defined peak prior to obtaining an excessively high flow value. Consequently, the stability and flow values are not always used and the selection of an optimum asphalt content is based exclusively on void criteria (AASHTO 1991, Bukowski 1991, and Stuart 1992).

Early SMA mix designs in the United States were influenced by the procedures used in Europe. Typically, mix designs in the United States have used the 50 blow Marshall and the 3 percent VTM criteria (Brown 1992). Recognizing the need for a standardized mixture design procedure, a working group was established by the Federal Highway Administration to develop model guidelines for SMA material, design, and construction (NAPA 1994). These SMA model guidelines for mixture design are given in Table 7.

SMA Mixture Properties

A comparison between SMA mixture properties and typical HMA properties is provided in Tables 8 and 9. A review of these properties indicates that the SMA stabilities are approximately 50 to 60 percent lower than the stabilities for HMA. Whereas, SMA flow values are between 14 to 70 percent higher than HMA mixtures. In a study by Brown (1992), a comparison was made between SMA and HMA mixtures in conjunction with indirect tensile, resilient modulus, gyratory, and confined creep laboratory testing (Table 9). That study found the SMA mixtures did not perform better than HMA mixtures in typical laboratory tests, such as, stability, flow, indirect tension, and resilient modulus (Figures 6 to 9), but the SMA performed at least equal or better in creep and gyratory testing (Figures 10 to 12). Laboratory test results indicate that SMA mixtures should have poorer performance potential than HMA mixtures. However, field studies show that SMA mixtures have provided excellent performance which implies that the typical laboratory tests used for HMA are not applicable to SMA and that tests more related to SMA performance are needed (Brown and Manglorka 1993).

SMA Production and Placement

Production

SMA mixtures can be produced using the same equipment and methods used for conventional HMA concrete. In Europe there are almost no drum-mixer plants, therefore their experience in producing SMA mixture is limited to batch

plants. In Europe, aggregates are generally single or narrowly sized (van der Heide 1992). Whereas, in the U.S. an aggregate stockpile will normally contain several (usually at least two or three) different sizes of aggregates. European plants generally have many more separate cold feed bins than their U.S. counterparts. The European batch plants usually have six hot bins versus four for the U.S. plants. These features allow the Europeans closer control of the aggregate gradation and the flexibility for rapid changes in the mixture produced. This is needed because they generally deal with small production runs of particular mixes and will normally produce several different mixtures each day. Drum-mix plants, which are prevalent in the U.S., have been used to produce SMA mixtures in this country (Scherocman 1992).

Most European batch plants have several separate cold feed systems in place to add mineral filler to the mixture. This is because many of their asphalt concrete mixtures, not only SMA, require substantial amounts of mineral filler (van der Heide 1992). Mineral filler is not normally added to most U.S. asphalt concrete mixtures. Therefore, U.S. batch plants may not have this capability or if they do it is usually limited to only one feed system for mineral filler. U.S. batch plants generally have the capability to add bag house fines into the mixtures. European batch plants also have this capability, but it is seldom used.

In Europe the use of additives such as: organic (cellulose) fibers and occasionally mineral fibers and polymers for SMA is more predominate and most plants are equipped with silos and auger feeds to provide these materials as needed. At plants not normally producing SMA the additives are typically added manually directly into the pugmill during the dry-mix cycle. Fibers that are added this way are normally supplied in meltable plastic bags, proportioned for the size of the batch being produced (van der Heide 1992). The majority of U.S. batch plants would probably have to add fibers manually as described above.

The time of dry mixing for dense graded asphalt concrete mixtures in Europe is approximately 5 seconds after all the materials have been added to the pugmill. This time is extended from 5 to 15 seconds for SMA mixtures (van der Heide 1992). The mixing time is usually longer (ie, up to 15 seconds) when cellulose fibers are used. The wet mixing time is also usually extended for 5 to 20 seconds for SMA mixtures (Scherocman 1992). As with dry mixing, the greatest increase is required for those mixtures containing cellulose fiber as the additive. The use of polymer modified asphalt cements does not effect the dry or wet mixing times.

European mixing temperatures for dense-graded mixtures usually vary from 160 °C (320 °F) to 176.7 °C (350 °F). SMA is usually mixed at the lower end of this range to prevent drain off related to the higher temperatures (van der Heide 1992). These European mix temperatures are higher than would normally be found in the U.S. because of the use of harder asphalts.

to prevent asphalt drain off (Scherocman 1992 and van der Heide 1992). SMA mixes are usually transported to the job site and placed immediately after mixing. The adhesion caused by the high binder contents can result in the SMA sticking to the bed of the haul truck. Therefore, the truck beds must be in good condition and a suitable amount of release agent should be applied (Scherocman 1992).

U.S. construction experience. The major differences between conventional asphalt concrete mixtures and SMA mixtures is amount of mineral filler in the mixture. Some U.S. plants have existing capabilities to add mineral filler to the mixture. However, placement of the mineral filler into the mixture in some cases has required adaptations or the use of special procedures at the plant. The addition of the mineral filler has been accomplished in some cases through pneumatic conveyance to the weigh hopper for batch plants and into the drum for drum mixers. The mineral filler has also been added in premeasured meltable bags at batch plants.

The addition of fibers (usually cellulose) into the SMA mixture into batch plants is generally accomplished manually with the use of premeasured disposable bags. For drum-mix plants the use of pellets (50 percent fiber and 50 percent hard based asphalt cement) added partway down the drum-mixer is the method normally utilized. Adjustments to the mixing time are often based on a visual evaluation of the dispersion of the fibers throughout the mixture (Scherocman 1992).

It has been reported that it is difficult to do handwork or rake SMA mixture. Areas that are hand worked will be visually obvious after compaction (Scherocman 1992). Compaction should start as soon as possible after placement. Rolling is usually accomplished by steel-wheel rollers, although recently some initial rolling has been accomplished with vibratory rollers. Pneumatic-tire rollers are not used because they tend to pickup mixture due to the tackiness of the SMA mixture.

Placement

The placement equipment and procedures used in Europe are very similar to those used in the U.S. One major difference is that Europeans use combination pavers. These pavers use not only a vibrating screed but also have at least one and sometimes two tamping bars that work in combination with the screed. These are used in combination for intermediate or leveling course placement. To achieve a high degree of surface smoothness on surface courses contractors will use either the tamping bars or the vibrating screed but not both during placement. Therefore, in placing surface course the European and U.S. pavers perform in the same manner. The major difference in placing surface course mixes is that the Europeans generally operate their pavers at slow speeds of 3 to 4.6 m (10 to 15 ft) per minute compared to 9.1 to 18.3 m (30 to 60 ft) per minute in the U.S. The difference in speed can probably be traced mainly to the lower production rates of European versus U.S. plants (Scherocman 1992 and van der Heide 1992).

In Europe longitudinal joints for roadways are constructed in one of two ways. The first and preferred method is to run pavers in echelon and form a hot joint. The second method, where a cold joint is formed, is to use a bitumen tape which is placed along the cold joint and the heat of the new mix that is placed melts the tape to seal the joint (van der Heide 1992). Except for the bitumen tape joint construction practices are similar between the U.S. and Europe.

Compaction equipment and practices are similar between the U.S. and Europe. Density requirements in Europe are greater than those normally specified for roadway mixes in the U.S. although they are similar to those specified for airfield pavements. Density is typically specified as a percentage of Marshall density or as a percentage of theoretical maximum density (van der Heide 1992). In European practice the compaction of SMA is accomplished by static steel-wheeled rollers. Pneumatic-tire and vibratory steel wheel rollers are not used to prevent pickup and breakage of the aggregate, respectively (Scherocman 1992 and van der Heide 1992).

3 Laboratory Evaluation

Evaluation Plan

The laboratory evaluation of SMA was conducted to compare its mixture properties to those of a control or standard airfield asphalt pavements. This involved the determination of the standard Marshall properties and an evaluation of the potential for the prevention of rutting and cracking and for its resistance to stripping. These last three properties were investigated through the use of indirect tensile and unconfined creep-rebound tests.

The first part of the laboratory evaluation involved determining the optimum asphalt content of the SMA for the gradation selected by 50 and 75 blow Marshall methods and by use of the GTM, see Table 10. Because of experience and previously available information concerning the control mixture, this mixture was compacted using only the GTM (Table 10). After the determination of the optimum asphalt content for both mixtures, a second set of specimens was made and evaluated using the GTM. Specimens were made at the optimum asphalt content and at the optimum asphalt content ± 0.5 percent (Table 11). During the compaction of these specimens on the GTM the following dada were recorded or determined: specimen height, Gyratory Shear Index (GSI), unit weight, roller pressure, and shear at 30, 60, 60, 120, 150, 180, 210, 240, 270, and 300 revolutions. Retained stability and maximum specific gravity tests were also performed. The final part of the evaluation was to perform indirect tensile and unconfined creep-rebound tests. These tests were run on the control and SMA mixtures at various temperatures and asphalt contents. Table 12 provides a complete listing of all tests that were performed.

Materials

Control (standard airfield) mixture

The control mixture used as a comparison for the SMA was a 19 mm (3/4 in.) dense-graded high-pressure (75-blow) Marshall mix design. The aggregate used was a crushed limestone aggregate with approximately 10 percent natural sand added. Table 13 lists the gradation used for the mixture.

The asphalt cement used was an AC-20, which met the requirements of ASTM D 3381.

SMA mixture

The SMA mixture had a gradation as given in Table 13. The aggregate used was a crushed limestone. The minus 75 μm (No. 200) sieve material was limestone dust. No natural sand was used in the mixture. The asphalt cement used as the binder in the SMA mixture was the same AC-20 as used in the control mixture. The SMA mixture also contained 0.3 percent cellulose fiber by total weight of mixture.

Sample Preparation

Mixing

A Univex rotary mixer with a wire mixing head was used to mix the aggregate and asphalt cement for each individual specimen. The mixing temperature was determined for the AC-20 binder according to ASTM D 1559, which requires a viscosity of the asphalt cement at the time of mixing of 280 $\pm$ 30 cst and resulted in a mixing temperature of 154°C (310°F).

Compaction

The specimens were compacted through the use of either the Marshall method or the GTM. The laboratory test specimens as outlined in Tables 10, 11, and 12 were compacted by one of these methods. The compaction temperature was determined for the AC-20 binder according to ASTM D 1559, which requires a viscosity of the asphalt cement at the time of compaction to be 170 $\pm$ 20 cst which resulted in a compaction temperature of 146°C (295°F). Both 50 and 75 blow Marshall specimens were fabricated for this study. The majority of the specimens that were manufactured for testing were made with 75-blow Marshall and corresponding GTM compaction forces to investigate the SMA mixture in comparison to a standard airfield mixture.

Testing

Marshall properties

The SMA mixtures were produced using three different compaction efforts. These were the 50-blow and 75-blow Marshall and the GTM set at 1,378.9 Kpa (200 psi) and 1.0 degree of angle (equivalent to a 75 blow

Marshall mix). The Marshall properties for each mixture produced by the different compaction methods were determined for comparison purposes.

Results of Marshall testing

The optimum asphalt content determined from the Marshall mix design of the SMA with 75-blow compaction effort was 6.0 percent by total weight of the mixture. The optimum asphalt content of the control mixture at the same compactive effort was 5.1 percent by total weight of the mixture. Table 14 lists the results of the initial Marshall tests on these two mixes at the various asphalt contents investigated. The 6 percent for the SMA mixture was selected based on the results from the GTM. One parameter that can be determined with the GTM is the gyratory stability index (GSI). The GSI parameter indicates a mixture's potential for plastic flow at the applied compactive effort (McRae and Foster 1959 and McRae 1965). A GSI value greater than 1.1 indicates that the mixture is susceptible to rutting under traffic. SMA specimens made with the GTM produced GSI values less than 1.1 for asphalt contents below 6.0 percent. SMA specimens made for various tests at the optimum asphalt content had an average void content of 2.9 percent. A retained stability test (U.S. DOD 1967) was conducted on samples prepared at the optimum asphalt content and they showed a retained stability of 100 percent, see Table 15.

Indirect Tensile Test

Background

The most straightforward approach to determine the tensile strength of a substance would be to construct a cylindrical/prismatic specimen of constant cross section and subject it to a direct pull. However such a technique would not be feasible for materials like asphalt concrete because of the difficulty involved in transferring tensile load to the specimen. It is difficult to the point of impossibility to grip a specimen for the purpose of applying an axial pull without generating stress concentrations and end bending moments that would induce premature failure and therefore result in an incorrect measure of tensile strength. Therefore an indirect method is used to determine tensile strength of asphalt concrete.

A circular disk subjected to concentrated diametrically opposing forces is shown schematically in Figure 13, and the state of stress in the disk is given by the equations (Frocht 1948),

$$\sigma_x = \frac{2P}{\pi t}\left[\frac{(R-y)x^2}{r_1^4} + \frac{(R+y)x^2}{r_2^4} - \frac{1}{D}\right] \tag{1}$$

$$\sigma_y = \frac{2P}{\pi t}\left[\frac{(R-y)^3}{r_1^4} + \frac{(R+y)^3}{r_2^4} - \frac{1}{D}\right] \tag{2}$$

$$\tau_{xy} = \frac{2P}{\pi t}\left[\frac{(R-y)^2 x}{r_1^4} - \frac{(R+y)^2 x}{r_2^4}\right] \tag{3}$$

where

P = applied load

t = specimen thickness

R = disk radius

r_1^2 = $x^2 + (R - y)^2$

r_2^2 = $x^2 + (R + y)^2$

D = disk diameter

On the y-axis where

x = 0

r_1 = R - y

r_2 = R + y

equations 1,2,and 3 become

$$\sigma_x = -\frac{2P}{\pi t D} \tag{4}$$

$$\sigma_y = \frac{2P}{\pi t}\left[\frac{2}{D-2y} + \frac{2}{D+2y} - \frac{1}{D}\right] \tag{5}$$

$$\tau_{xy} = 0 \tag{6}$$

From examination of equations 1 through 6, it is seen that across the vertical central section of the disk, that is, in the plane defined by the line of the loads, horizontal tensile stress, σ_x, is maximum and has a constant value and shear stress, τ_{xy}, is zero. However, vertical compressive stress, σ_y, becomes infinitely large directly beneath the loads, that is, at y = ± D/2R. Therefore, except for the non-uniform and, theoretically, infinite compressive stresses at the cylinder surface directly beneath the loads, diametrical compression of a disk is a simple and effective method of determining tensile strength.

Problems associated with concentrated/point loads are largely avoided (or at least controlled) in laboratory testing by loading the specimen through a curved strip of finite thickness. In this way, applied force is not concentrated but,

rather, is distributed over a small area. Consequently, compressive stress does not become so large at the point of load application that failure in compression occurs and dominates specimen behavior before tensile properties can be observed. Therefore, to determine tensile strength, an *indirect tension test* of the type described is performed through controlled axial deformation of a circular cylindrical asphalt concrete specimen along its central axis through a loading strip of finite width.

Indirect tensile test procedure

The indirect tensile test performed in this investigation was conducted according to ASTM D 4123. The specimen tested was a circular cylinder, 4.0 in. in diameter and approximately 2.5 in. in length, see Photo 1. The test configuration is shown schematically in Figure 14 where the specimen is loaded through rigid strips placed in solid contact with the periphery of the specimen (also as photo 1). From Figure 14, it is obvious that the axis of the cylindrical test specimen lies in the horizontal plane and the cylinder rests on an edge element. The curvature of the loading strips exactly matches that of the specimen. The force, P, is applied through the top loading strip and onto the surface/edge of the specimen at a constant rate of displacement. During loading, force and deformation are measured with electronic transducers and monitored through a computer based data acquisition system. Software for data acquisition was prepared at WES in the Airfields and Pavements Division specifically for monitoring, controlling and acquiring data during various asphalt concrete laboratory tests. The rate of strain applied to the specimens was 2.0 in. per minute; displacement was applied with a servo-hydraulic, closed-loop feedback system. Tensile stress developed within the specimen was computed from Equation 4, where σ_x is tensile stress that develops along the vertical section between lines of load application. During testing, specimens typically crack and separate along the central axis of the specimen defined by the load application points.

Specimen response

Tensile stress versus deformation relationships observed in the indirect tension tests performed in this investigation have a very characteristic shape. Tensile stress (computed using Equation 4) initially increased rapidly and essentially linearly as vertical deformation was applied. Slope[1] of the tensile stress-deformation relationship is steepest during initial loading (although due to problems probably associated with seating, the slope or modulus may increase slightly in some specimens immediately after the start of loading). As tensile stress in test specimens increases to approach peak, stress-deformation behavior departs from linearity; slope of the tensile stress-deformation relationship begins to decrease rapidly as maximum load is reached and behavior

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[1] "Slope" refers to the average tangent if the curve obtained from plotting the change in tensile stress with change in applied edge deformation.

becomes erratic and unpredictable. After decreasing to zero at peak load, slope of the tensile stress-deformation relationship becomes negative and the value of tensile stress decreases steadily and rapidly as deformation is further increased. Asphalt concrete fails progressively after peak stress and becomes increasingly less able to support tensile stress. Ultimately, measured tensile stress decreases either to zero or some residual load that is a very small fraction of the peak stress.

Typically, tensile stress induced in an asphalt concrete specimen during a indirect tensile test literally splits the specimen apart; the specimen separates and void spaces may open up in the region along the line of loads. In some tests at large displacement, a portion of a geometrically distorted specimen will, by chance, come into contact with the load head and cause a rise in force registered by the load sensor (due to compression of that block of material). The indication of this increase in force in terms of the data generated, is an increase in tensile stress. However, this is obviously a false indication because it does not reflect the ability of the test specimen to produce tensile stress. At this point in the test, that region in the specimen where tensile stress develops in response to applied (diametrically opposing) loads has failed and has probably been destroyed. An increase in tensile stress past the peak load is false and should be ignored.

Discussion of indirect tensile test results

The indirect tensile tests performed in this investigation allowed for a direct comparison of performance between the SMA and control mixtures at various asphalt contents and temperatures. Test specimens were constructed at optimum asphalt content and at contents 0.5 percent above and below optimum for both mixtures. Tests were also conducted at temperatures of -17.8, 25, and 40°C (0, 77, and 104°F). Thirty-four indirect tensile tests were performed during the investigation to compare the tensile behavior between the SMA and the control mixtures. The physical properties of the specimens tested for tensile properties are shown in Table 16. The test results are summarized in Table 17. Tensile stress-deformation relationships for all tests performed are presented in Appendix A. Evaluation of the results listed in Table 17 provide the following observations:

a. There was a notable decrease in tensile strength with an increase in temperature for both mixtures.

b. The deflection at maximum load generally increased with increasing temperature for both mixtures. For the control mix there seemed to be a peak in deflection at maximum load at 25 °C (77°F).

c. The elastic modulus decreased with increasing temperatures for both mixtures.

Unconfined Creep-Rebound Tests

Background

Creep may be very simply defined as progressive plastic flow of a body in response to constant and sustained stress, the rate of flow being determined by the magnitude of stress applied and rheologic properties of the material comprising the body. Flow due to creep will result in gradual distortion and loss of efficiency and integrity in structures where continued good function and performance depends on a specific geometric shape. Susceptibility to creep, therefore, is an undesirable characteristic in any construction material. Asphalt concrete is a material that is susceptible to creep because it is a mixture of aggregates and asphalt cement. Asphalt cement is a highly viscous material at ambient temperatures the decreases in viscosity as the temperature increases. Because of this effect, the creep characteristics and strength of asphalt concrete are strongly affected by temperature. Because susceptibility to creep is an undesirable characteristic, it should be minimized; consequently, a test procedure to identify and quantify creep in asphalt concrete mixes must be devised.

To most clearly understand and quantify creep, it is important to study the phenomenon (as well as the ability of the test material to recover from it) in a uniform stress field. A uniform stress field exists, for the most part, in the middle of a long prismatic specimen of constant cross section. Asphalt concrete test specimens produced for this and other similar investigations are typically constructed in the form of cylindrical disks which are four inches in diameter and about two and one-half inches in thickness. Circular asphalt test specimens with a thickness, t, to diameter, d, ratio ($t/d = 2.5/4.0$) less than 1 are considered short circular cylinders; analyses have been performed (Balla 1960) which show that a uniform stress field does not exist in short circular cylinders that are subjected to axial compression applied by rigid end platens.

Saint Venant's principle, which states that stress uniformity in circular cylindrical test specimens can be achieved provided a satisfactory distance between constrained end conditions is obtained. In triaxial tests on solid circular cylinders, it is customary to consider that a thickness to diameter ratio of about 2 to 1 is adequate for routine testing (Wright, Gilbert, Saada 1978). Therefore, to achieve a more uniform stress and strain condition for the study of creep in asphalt concrete, the height to diameter ratio of normally constructed asphalt concrete test specimens was accomplished by stacking three 6.4 cm (2.5 in.) thick specimens to form a single creep test specimen, see photo 2. The result is a specimen with a total thickness to diameter ratio of about 7.5 to 4; this is believed to be adequate to minimize the effects of end friction and boundary stress transfer and therefore produce a reasonably uniform stress field. Creep was measured over a one inch length in the middle of the (geometrically) modified asphalt concrete specimen at the point that is the greatest distance from the ends and where stress and strain uniformity may be argued to be greatest.

Creep-rebound test procedure

ASTM has not issued a formal test standard for evaluating creep susceptibility at the time of this report. Therefore, the creep tests conducted for this investigation were performed using a test procedure devised at WES in the Airfields and Pavements Division, see Photo 2. Table 18 provides the physical properties of the specimens used for the creep-rebound tests. The tests were performed under the following conditions:

a. Allowing test specimens to equilibrate in an environmental chamber maintained at either 25° or 40°C (77° or 104°F) for a period of 24 hours.

b. Mounting two (diametrically opposed) vertical linear variable differential transformers (LVDT's) on a frame clamped to specimens at mid-height. (It should be noted that the axis of the cylindrical specimen was in the vertical direction in creep-rebound tests). The LVDT's and frame were mounted such that vertical deformation was measured over one inch of specimen height during loading and unloading.

c. Applying a seating load, or a preload to the specimens. Specimens tested in the environmental chamber at temperatures of 25° and 40°C (77° and 104°F), were seated with preloads of about 222.4 and 89.0 N (50 and 20 lb) ((27.6 and 11.0 KPa)(4.0 and 1.6 PSI)), respectively, which were allowed to remain on the specimens for a period of five to ten minutes before application of the test load.

d. Applying a test load to the specimens (in excess of the seating load). Specimens tested in the environmental chamber at temperatures of 25° and 40°C (77° and 104°F) were loaded with axial pressures of 275.8 and 103.4 KPa (40 and 15 PSI), respectively. The loads were allowed to remain on the specimen for a period of one hour. Vertical deflection data were observed at elapsed time increments after load application of 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, 900, 1800, and 3600 seconds.

e. Removing all load (test load as well as the seating load) from the specimen and observing rebound. Rebound deflection data were observed at the same time increments after load removal as stated above for a total time of test (when combined with the loading period) of two hours.

Variation in measured creep response

Reasonable steps were taken in the investigation of creep response to avoid, or at least minimize the effects of cap and base end restraint (due to friction) on observed load-deformation behavior in the test specimen. The middle third of the test specimen is used because it is as far from the ends as physically possible, so Saint Venant's principal ensures maximum stress and strain uniformity

in the section of the test specimen where the response is measured. However, even though steps are taken to ensure maximum stress and strain uniformity, spatial density uniformity and structural homogeneity are both functions of specimen construction/preparation technique and density uniformity and structural homogeneity cannot be ensured from specimen to specimen.

Creep response in asphalt specimens tested at the stress and temperature levels enforced during this investigation typically took place at small displacement amplitude, and under applied compressive loading that was small relative to the compressive strength of the specimens. For example, measured deformation was often less than 0.02 cm (0.008 in.) over the 2.54 cm (1 in.) of control height; this amount of deformation converted to (vertical) strain is 0.8 percent, which is a relatively small magnitude of strain. Stiffness and/or creep response of an asphalt concrete specimen will be a function of the number of aggregate to aggregate contacts in the test specimen since the aggregate skeleton effectively carries applied load. A different number or distribution of aggregate to aggregate contacts from specimen to specimen will mean that a different portion the applied load will be carried by the bitumen (from specimen to specimen); since bitumen is a (viscous) liquid, if it is required to carry a greater load during a given test, creep response will be more pronounced. It is almost certain that aggregate to aggregate contact conditions are statistically different between specimens because of chance differences in aggregate placement during specimen preparation. These chance differences in placement cause differences in structure and homogeneity in prepared specimens and result in measurable differences in response to loading in spite of care and effort spent to ensure identical test specimens. Therefore, variation in measured values of creep deformation from specimen to specimen is inevitable and must be expected. More precise and repeatable quantification of creep response might be expected if large deformation levels resulted from the loading. At large deformation levels, the number of aggregate to aggregate contacts approach a limiting value, increasing the reliability of the test[1]; however, at small deformation levels, creep response is dictated by chance specimen placement.

Conditions other than specimen preparation may also influence creep response. Since the value of induced deformation is small, placement of the instrument frame on the test specimen may have a significant influence on the way displacement is measured and therefore on indicated creep response observed in this investigation. Because no two specimen peripheries are the same, the instrument frame cannot be fixed to different specimens in exactly the same way, therefore measurement variation and uncertainty from specimen to specimen will result.

In summary, variation in measured values of deformation which are used to quantify and characterize creep in asphalt concrete specimens suggest that some

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[1] Similarity in stress-strain characteristics at larger deformation levels may be seen in the tensile test specimens. The modulus and strength values observed in tensile tests conducted on "identical" specimens is statistically good. Conversely creep deformation response on "identical" specimens shows more statistical variation.

of the factors that influence the response may not be adequately taken into account. These factors may include those described above, notable:

a. Random differences in density uniformity and structural homogeneity due to specimen preparation. The precision and repeatability with which specimens are formed at the level that controls creep response may be low.

b. The fact that creep response is determined by small displacements that do not effectively alter the structure of specimens on which creep tests are performed.

c. There may be difficulty involved in placing the instrument frame with adequate precision to ensure measurement repeatability from test to test, especially considering that deformation levels determining creep response are small.

Specimen response

Under an applied static load, asphalt specimens tested in this investigation deformed under loading by decreasing in vertical height monotonically with time. Compressive deformation rate was greatest immediately after loading and decreased (approximately) logarithmically with time as will be demonstrated later. When the load was removed, the test specimens rebounded to lengthen monotonically and, like the response under compressive load, rebound deformation rate decreased logarithmically with time.

Parameter to quantify creep and rebound

In order to compare propensity to creep (or rebound) in this investigation, it will be necessary to devise or define a parameter that quantifies creep. Observation of the shape of the creep and rebound relationships (typical results are shown in Appendix B) suggests that a power curve (i.e., a plot of the logarithm of time against the logarithm of deformation), would provide a suitable linear fit of the data. When creep and rebound data were curve-fit with a power curve, it was discovered that in all but a few cases the coefficient of the correlation of the curve-fit was greater than 0.9 and in many cases, substantially greater; such high coefficients of correlation suggest that a good approximation for the relationship is, indeed, a power function. Power function equations are of the form:

$$\Delta = a(T^b) \tag{7}$$

where

Δ = specimen deformation in centimeters

T = time after the start of creep or rebound in seconds

a = curve-fit coefficient associated with intercept of the fitted line

b = curve-fit coefficient associated with slope of the fitted line.

Coefficients of the power function equations are given in Table 19 for all creep and rebound tests performed, along with the coefficients of correlation, R^2, for the individual curvefits.

The actual log-log plots and the resulting power function curve-fits for all the tests performed (i.e., creep as well as rebound) may be seen in Appendix C. Since there is good (power function curve-fit) correlation for creep and rebound data, the parameter, b, defined above will be used as the index to quantify and compare the propensity for creep or rebound. This is a reasonable and sensible selection because b is effectively a slope and the propensity for creep is directly proportional to the value of this (slope) coefficient. As the value of b increases and decreases there is a greater and lesser tendency, respectively, of a specimen to creep or rebound. Another argument for using b as an index for quantifying creep characteristics is that the interval of time over which b was determined and defined was constant in this investigation. All specimens tested were allowed to creep and rebound for a period of 3600 sec and values of b were computed over that time interval.

It should be mentioned that the value of a, the coefficient associated with deformation intercept, is more arbitrary and is more difficult to interpret physically with creep (and rebound) characteristics than b.

Discussion of creep-rebound test results

Twenty-five creep-rebound tests were performed in the investigation to compare the creep behavior between the SMA and control mixtures. These asphalt concrete mixtures were prepared at different test temperatures, compaction efforts and asphalt contents. The test conditions are summarized on Table 19. The deformation versus time relationships for all creep and rebound tests performed in this investigation are shown in Appendix B.

Control versus SMA

In five tests conducted at the optimum asphalt content of each of the mixes, three tests were conducted on the control mix and two on the SMA mix. Tests results are shown under the "K" series[1] in Table 19 and were performed on specimens compacted by gyratory compaction and tested at 25°C (77°F), under an axial pressure of 275.8 KPa (40 PSI). The optimum asphalt contents of the control and SMA mixes were 5.1 and 6.0 percent respectively. Analysis of the b coefficients shows that the average b for the three control tests were

[1] "J", "K" and "L" series test specimens were molded at asphalt contents which are one-half percent less than optimum, at optimum, and one-half percent greater than optimum, respectively.

0.1625 and -0.0366 for creep and rebound, respectively. Conversely the (average) values (of b) for specimens of the SMA mix were 0.3100 and -0.0336 for creep and rebound, respectively. If it is acceptable to use the b coefficient as the basis for comparison, then the conclusion reached is that, for the molding and test conditions used, specimens composed of the SMA mix statistically creep about twice as much (about 90 percent more), and rebound a little less (about 8 percent less) than specimens of the control mix.

Gyratory versus Marshall Hammer compaction

Six tests were conducted on specimens prepared from the SMA mix to compare the effects of compaction procedure. Three of the specimens were compacted by gyratory compaction and three using the 50 blow Marshall Hammer and are shown in Table 19 as the "SMA-K" series tests and the "H50K" series tests, respectively. All test specimens in those series were prepared at an asphalt content of 6.0 percent, were tested at a temperature of 40°C (104°F) and were loaded with an axial pressure of 103.4 KPa (15 PSI) (in addition to the seating load of about 11.0 KPa (1.6 PSI)). Analysis of the b coefficients for these tests showed that average values for the Marshall Hammer compacted specimens were 0.2853 and -0.0219 for creep and rebound, respectively. Average values of the b coefficient for gyratory compacted specimens were 0.1553 and -0.0269 for creep and rebound, respectively. This means that, based on analysis of the specimens tested, the Marshall Hammer compacted specimens statistically creep about 80 percent more and rebound about 20 percent less than specimens produced by gyratory compaction.

Creep and rebound variation with asphalt content

The effect of asphalt content on creep was investigated in this study by testing specimens at the optimum asphalt content as well as one-half percent above and below optimum. SMA and control mix specimens were molded and tested under the same conditions so that creep characteristics of the two mixes could be compared directly. Test specimens were formed by gyratory compaction, were loaded with an axial pressure of 103.4 KPa (15 PSI) and were tested at a temperature of 40° (104°F).

Control mix performance

The optimum asphalt content of the control mix was determined to be 5.1 percent; two tests were performed at an asphalt content of 4.6 percent and three tests each were performed at asphalt contents of 5.1 and 5.6 percent. Propensity to creep was quantified in terms of the b coefficient defined above.

Figures 15 and 16 show b coefficient versus asphalt content for creep and rebound tests, respectively, performed on specimens of the control mix. Although there is scatter in the data indicated by somewhat low coefficients of

correlation (R^2 = 0.134 for creep tests and 0.295 for rebound tests based on linear models) the statistical trend is for the propensity for creep to increase as asphalt content increases over the range investigated. Similarly, the propensity for rebound is to increase as asphalt content increases.

SMA mix performance

Optimum asphalt content of the SMA mix was determined to be 6.0 percent and three tests each were performed at asphalt contents of 5.5, 6.0 and 6.5 percent. Figures 17 and 18 show b coefficient versus asphalt content for creep and rebound tests. The statistical trend for the propensity to creep is similar to that of the control mix (based on slope of the asphalt content versus b relationship). However, the tendency to rebound decreased in the SMA mix over the range of asphalt contents covered by this investigation.

It should be noted that density of the aggregate skeleton is very constant for the specimens and mixes prepared and tested in this study. Variation in density of the aggregate skeleton (that is, density computed with bitumen removed) was so slight ($\pm$6.4 kg/m^3 ($\pm$ 0.4 PCF) for the control mix and 14.4 kg/m^3 ($\pm$ 0.9 PCF) for the SMA mix) that no correlation could be found between b coefficient and aggregate density. Additionally, it should be mentioned that density of the aggregate skeleton is of the order of 2,338.9 kg/m^3 (146 PCF), which is a very high density and indicative of a very strong material in its own right. The implication of the high and nearly constant aggregate density achieved in specimens of this investigation is that aggregate density has been effectively eliminated as a variable in this investigation of propensity to creep.

Variation and scatter were observed in the relationships chosen to quantify creep. In fact, the linear model selected to relate propensity to creep (in terms of the b coefficient) with asphalt content, explained only about ten percent of the creep data and about 30 percent of the rebound data. This means, of course, that factors which were not adequately taken into consideration accounted for and explained the remainder of the data. These factors would likely include (but not be limited to) density inhomogeneity, stress- and strain-inhomogeneity, variation in air content of the mixes under test, and variation in the character and distribution of voids and solids in the test material, to name a few. Factors like those listed may, in fact, be so uncontrollable that it may not be possible to satisfactorily study and formulate them in a systematic manner, even in a well controlled laboratory environment. One of the purposes of a laboratory study (like the present one) is to define tolerance ranges in the dependent variable (such as creep susceptibility), because if factors which significantly influence material behavior are uncontrollable under laboratory conditions, it is likely that such factors are at least as uncontrollable under field conditions. A first step in managing or improving a property of interest in a material under evaluation is to identify factors which control behavior associated with that property. Another purpose of a laboratory study is to compare one material against another to determine which one is more advantageous in terms of desired characteristics. Neither improvement of material properties nor comparison of one material with another can be accomplished without a laboratory investigation. Additionally, it is possible that characteristics of a

material under investigation are such that properties of interest cannot be economically controlled or improved under any circumstance. Such a determination that can be made only with a well designed and executed laboratory study.

4 Field Demonstrations

Edwards AFB, CA

The demonstration project at Edwards AFB, California, was located at the intersection of Lancaster Boulevard and Wolfe Avenue. A traffic light was added to the intersection and the roadways were widened through the addition of turning lanes. Refer to Figure 19 for the location and general layout of the demonstration project. The main portion of the SMA was placed as a 50.8 mm (2 in.) overlay of existing asphalt concrete. Where the SMA was to tie back into the existing pavement, the pavement surface was milled to a depth of 5.1 cm (2 in) and then feathered back to the original surface within approximately 7.6 m (25 ft), see Photo 3. The remainder was placed in the reconfigured intersection with 76.2 mm (3 in.) of SMA placed directly on a base course (Photo 4). All paved surfaces were designed with a minimum of 2 percent cross slope. The major distresses observed on the existing asphalt concrete pavement prior to overlay were longitudinal and transverse cracking. These cracks were sealed with a hot-applied, asphalt-rubber sealant prior to the application of the SMA overlay. Several utility cut patches existed throughout the section (Photo 5).

The pavement area overlaid on Lancaster Boulevard was approximately 732 m (2400 ft) long and 9 m (30 ft) wide. Approximately 137 m (450 ft) of Wolfe Avenue was also overlaid with SMA. The reconfigured intersection added approximately 1858 square meters (20,000 square feet) of surface area to the paving of the two roads. The SMA pavement was placed on the 5th and 6th of August 1993.

Plans and specifications

The demonstration project was developed by taking an existing HMA overlay project and substituting a SMA mixture for the wearing course. The original HMA specifications for the project referenced the following sections of the California Department of Transportation (Caltrans) 1988 standard specifications.

a. Section 6 - Control of Materials.

b. Section 39 - Asphalt Concrete.

c. Section 92 - Asphalts

A 50-blow Marshall mixture design was specified along with stability, flow, and void requirements listed in Table 1. The voids were to be determined by ASTM D 2041 or MIL-STD-620A, method 101 using the apparent specific gravity. A retained stability of 75 percent as determined by MIL-STD-620A, method 104 was required. The changes made for SMA construction included increasing the batch mixing time for an additional 10 to 30 seconds and using only steel-wheel rollers to compact the SMA mix to 94 percent of the maximum theoretical density.

The changes or adaptations required for the materials used in the SMA included the following:

a. The aggregate gradation and limit tolerances.

b. Aggregate requirements for flat and elongated particles, soundness, and fractured faces were added to the specification.

c. Mineral filler conforming to ASTM D 242 was required. A maximum of 20 percent could be smaller than 0.02 mm and this material could not have a plasticity index of not greater than 4.

d. A cellulose fiber was used in the SMA mixture at a rate of 0.3 percent by weight of total mix.

Mix design procedure

General. The Marshall mix design was performed in accordance to ASTM standards. The percentages of each material used to make the SMA mixture are listed in Table 20. The method used for compaction of Marshall samples in the project specification was ASTM D 1559, which requires a compaction temperature such that the viscosity of the asphalt cement is 280 ± 30 cst. This ASTM test method also specifies a mixing temperature that provides a viscosity of the asphalt cement of 170 ± 20 cst. Based upon asphalt cement viscosity tests, the mixing and compaction temperatures were determined to be approximately 141 ± 2.5°C (285 ± 5°F) and 149 ± 1.5°C (300 ± 3°F) respectively. The standard Corps of Engineers (CE) procedure for laboratory compaction of Marshall samples is to compact at 121 ± 2.5°C (250 ± 5°F) according to Mil Std 620 A. Samples were compacted at this temperature during the initial mix design process. Additional samples compacted at 141 ± 2.5°C (285 ± 5°F) did not achieve a higher density than those compacted at 121 ± 2.5°C (250 ± 5°F).

Materials. The contractor submitted the following aggregates for use in the SMA mix:

a. 19 mm (3/4 in) crushed stone.

b. 12.7 mm (1/2 in) crushed stone.

c. 9.5 mm (3/8 in) crushed stone.

d. Sand.

e. Fly Ash.

These aggregates were blended together to obtain a gradation which satisfied the limits given in Table 20. The tests conducted on the aggregates (a through d above), included gradation, specific gravity, absorption, soundness, and flat and elongated particles. Tables 20 and 21 provide the results of the tests performed on the aggregates. The fly ash met ASTM requirements as given in the specification.

An AR-4000 asphalt cement was submitted as the binder for the SMA. The asphalt cement met all requirements for an AR-4000 as provided in ASTM D 3381. Cellulose fibers or cellulose pellets were required by the project specification for use in the SMA mix. The project specification requirements for each type of cellulose material followed manufacturer's recommendations and are given in Table 22. The amount of fibers added to the mixture was required to be 0.3 percent of total weight of the mixture. The fiber supplier provided the contractor with premeasured plastic bags meeting this weight requirement for each 2,268 kg (2.5 ton) batch produced at the batch mixing plant.

Procedure and analysis. A series of specimens were made, with three specimens at each of five different asphalt contents. The specimens were each compacted with 50 blows from a standard hand hammer as described in ASTM D 1559. The specimens were compacted at 121 $\pm$ 2.5°C (250 $\pm$ 5°F) which is the normal temperature for the standard CE procedure. The results of these tests were then used to select an asphalt content. Six additional specimens were fabricated using the optimum asphalt content and a compaction temperature of 141 $\pm$ 2.5°C (285 $\pm$ 5°F) as specified in ASTM D 1559. Three of these specimens were tested for retained stability as specified in Mil-Std 620, Method 104. The test results of the samples prepared according to ASTM D 1559 and used as the job mix formula (JMF) are given in Table 20. The JMF met all specification requirements except for stability, 6,236 N (1,402 lb) versus 7,117 N (1,600 lb). The Marshall stability value was lower than required in the specifications but it was determined to not be critical enough to reject the mixture. The reason that the low stability value was determined to not be critical was because the controlling factor in the design was based largely on voids in the compacted mixture instead of stability and flow values. The typical plots used in the Marshall mix design for stability and flow to determine the optimum asphalt content were not greatly affected by changes in the SMA asphalt content. The optimum asphalt content as determined by the

Marshall testing was 7.3 percent by weight of the total mix. This asphalt cement content was then used for testing at the ASTM D 1559 temperature and for retained stability acceptance. Comparing these specimens to the calculated maximum theoretical specific gravity resulted in an air void calculation of 3.5 percent. Previous laboratory testing of SMA mixes had shown that determining the maximum theoretical specific gravity according to ASTM D 2041 can result in specific gravities lower than those obtained through calculations. Therefore, the calculated voids would be lower than the actual voids in the mixture. The asphalt content used for the JMF was reduced to 7.2 percent in an effort to keep the voids in the mixture closer to 4.0 percent. The results of the retained stability test showed a retained stability value of 94.6 percent, which exceeded the minimum requirement of 75 percent (see Table 20).

Construction

Plant. The SMA was produced in a batch plant. The plant was located in Palmdale, California, about 45 minutes from the job site. The mixing procedure used followed generally the same procedures normally used for batch plants mixing conventional HMA. The aggregates were weighed and batched from four separate hot bins. The mineral filler, which in this case was fly ash, was metered in from a silo through a screw auger. The AR 4000 asphalt cement was added to the pugmill two seconds after the start of the dumping of the aggregates into the pugmill. The exception from the normal procedures was that a premeasured bag of cellulose fibers was added manually to the pugmill at the same time that the aggregates were dumped into the pugmill. Photo 6 shows a pallet of the fibers placed on a walkway above the pugmill. From here the fibers were placed directly into the pugmill. The cellulose fibers were in premeasured plastic bags which disintegrated when exposed to the heat of the aggregate. A mixing cycle lasted 45 seconds from the start of dumping of aggregates to discharge of the completely mixed SMA.

Plant calibration. During the production of the initial batch of the SMA mixture, it was noted that there was a constant overflow and discharge of bin Number 1 which contained the fine aggregate. This indicated that an excess of fines were flowing into the plant. It was decided to calibrate the plants aggregate cold feeds. In conjunction with this, the stockpiles were sampled and a sieve analysis was conducted on the samples to verify the gradations obtained from samples submitted by the contractor. Table 24 contains the gradations of the stockpile samples obtained. These gradations were very close to the original gradations, only slightly coarser. At this time, the aggregates were being fed into the plant directly from large stockpiles using long feed belts. Attempts to calibrate the cold feed were very difficult because materials from each stockpile could not be completely stopped and restarted without coordination with the nearby crushing and stockpile control room.

The plant operator offered to feed out of the auxiliary cold feed bins which could be completely stopped and therefore accurately calibrated. A total of four auxiliary cold feeds were used, one each for the 4 stockpile materials

(Photo 7). The manually controlled gates beneath each bin were the only method available to control the amount of each material added. Small feeder belts under the gates were equipped with constant speed motors, preventing their use for controlling or varying the amount of material fed. These small feeder belts dumped the aggregate directly onto a large belt which carried the aggregates to the drier. The plant was calibrated to run at 127 Mg (140 tons) per hour and the gate feeds for each bin were set to supply this amount of material. After the calibration was completed, the aggregates were run through the plant, hot bin samples were taken, and new percentages of each hot bin were calculated to obtain the desired mixture. Table 25 provides the hot bin gradations and the percentage of each bin used to obtain the required JMF gradation based on these aggregate gradations.

One 2,268 kg (2.5 ton) batch was produced in the plant and a sample was obtained and evaluated. Test results obtained from this mixture are provided in Table 26. The test results indicated that the SMA mixture met the requirements given in the specification and approval was given to place the SMA mixture on the job site.

Adjustments to the job mix formula. The initial JMF based on stockpile gradations as provided to the contractor is given in Table 20. The amount of fly ash required for the initial JMF was 8 percent; however, after plant calibration the adjusted gradation required that the amount of fly ash be increased to 9 percent. One additional change that was made to the initial JMF was that the asphalt cement content was lowered from 7.2 to 7.1 percent by weight of total mix. Table 26 provides the test results of the SMA mixture test batches made using the adjusted JMF gradations. These results were then used as the JMF requirements for the SMA mixture placed for the demonstration project.

Placement of the SMA mixture began with an intermediate layer placed over a prepared base course in the reconfigured intersection of the demonstration project (Photo 8). The final adjusted JMF was further evaluated during the placement of this SMA. The mixture placed as the intermediate course met the requirements of the JMF.

A series of trial batches were made over a two day period prior to paving the demonstration project. As a result of these trial batches, minor adjustments were made to the JMF. A SMA test section was placed as an intermediate or leveling course on the base course in the reconfigured intersection area of the actual job site. The mixture met the specification requirements and became part of the final SMA pavement. The laboratory test results of the field test section are provided in the Sample Number 1 column of Table 26.

Laydown. Approximately 1,360.8 Mg (1,500 tons) of SMA were placed for the demonstration project over the two day period. Prior to placing the SMA, a tack coat of emulsified asphalt was applied. A conventional paver was used to place the SMA mixture. The mix was brought to the site in 22.7 Mg (25 ton) belly-dump trucks which windrowed the SMA mixture in front of the paver (Photo 9). An elevator attached to the front of the paver picked up the mixture and placed it in the paver hopper (Photo 10).

The paver had a 3.7 m (12 ft) wide screed with hydraulic extensions on either side to widen or narrow the paving lane as required. The outbound (south) lane was placed south to north using automatic grade control through a ski on the centerline side of the roadway. The outside edge was manually controlled to obtain a 50.8 mm (2 in.) thick overlay. The inbound (north) lane was also placed south to north.

Compaction. The SMA was compacted with a 9.1 Mg (10 ton) steel-wheel roller (Photo 11). During the first day of paving, the contractor applied only two coverages (equal to one forward and backward pass) to the SMA for compaction. The roller operator said that with additional passes he would start to feel movement of the mixture and felt that additional passes would result in a lower density. Test results from cores taken of the first days paving showed that the density achieved was just slightly under that specified: 93.3 versus 94.0 percent of maximum theoretical density. With these results the contractor had the roller operator apply 4 to 6 coverages on the mixture placed the second day in an attempt to achieve a higher density. The effectiveness of this additional rolling could not be determined because no cores were taken for testing by the contractor or base personnel.

An 7.3 Mg (8 ton) tandem steel-wheel roller was used as a finish roller to remove any remaining roller marks in the SMA surface. Although, it was lighter in total weight, the smaller width and diameter of the steel wheels appeared to cause this roller to impart more pressure to the pavement surface than the heavier roller. If the tandem steel-wheel roller was operated too close to the paving operation where the pavement was still hot, it would cause the surface to shove. The 9.1 Mg (10 ton) steel-wheel roller did not cause the SMA to shove.

Opening to traffic. The SMA pavement was opened to traffic within approximately 2 hours after completion of final rolling. Due to existing high-air temperatures and sunshine, the SMA pavement was not cooling fast; therefore, the contractor decided to spray the completed pavement with water with his water truck (Photo 12). This helped to cool the pavement surface and when the pavement was opened to traffic it was not damaged.

Performance

The SMA pavement was inspected after 1, 2, and 3 years of service. The pavement had experienced bleeding, at least to some degree, in all wheel paths but the bleeding had not caused any reported problems to date (Photos 13 and 14). The bleeding did appear to be greater in the braking and turn areas. No bleeding was evident in the nontraffic area formed as a turn lane. The bleeding appeared to be worse on the southbound lane versus the northbound lane. One potential reason for the increased amount of bleeding on the southbound lane would be the traffic pattern. The inbound commuter traffic flowing north would be heaviest during the morning hours when the pavement is the coolest. The outbound commuter traffic flowing south would be heaviest during the late

afternoon when the pavement temperatures are higher. There are only a few isolated areas of excessive or ponded asphalt cement on the surface. The bleeding in the remaining locations does not completely cover all surface aggregate, but tread marks have been left in many of the locations. To date, no reflective cracks have appeared in the SMA section. After three years there were several barely noticeable transverse lines across the paving lanes throughout the entire area. These lines appear to have been caused by excess asphalt on the surface probably caused by stopping and starting of the paver screed (Photo 15).

The wheel paths were checked for rutting using a straightedge. Little or no rutting was detected. Some small deviations from grade were measured throughout the pavement area but these were generally within construction tolerances (Photo 15). The only areas that exceeded these tolerances were near the intersection of Lancaster Boulevard and Wolfe Avenue where vehicles stopped for the light (Photo 16). There was also a large deviation in grade along the eastern edge of Lancaster Boulevard where the pavement structure was widened. Along this edge where trafficking had occurred, there was a drop off of approximately 12.5 mm (0.5 in.) in the new pavement structure. The exact cause of the 12.5 mm (0.5 in.) drop off was not determined; however, it was assumed that it was probably due to densification of the new base course under traffic. The existing pavement surfaces were also checked for rutting and not appreciable amounts could be detected (Photo 17).

Traffic count information obtained in 1994 for the planned rehabilitation of the remaining length of Lancaster Boulevard revealed an average daily traffic (ADT) rate of approximately 2,000 vehicles, in each direction. Vehicle counts generally indicated a slightly higher volume of southbound traffic versus northbound traffic. The northbound counter was placed on Lancaster Boulevard north of the traffic light at Wolfe Avenue. The southbound counter was placed on Lancaster Boulevard several miles further south, before the south gate. The ADT data showed of the 4,000 vehicles that normally use the road, that 75 to 80 percent were passenger cars with two and three axle single-unit trucks making up the largest percentage of the remaining traffic.

Laboratory evaluation of field samples. Six core samples of the SMA pavement were obtained from Lancaster Boulevard for analysis in August of 1994. The cores ranged in thickness from 49.2 mm (1.94 in.) to 52.4 mm (2.06 in.). The core samples were taken from both the wheel paths and between wheel path locations on the southbound lane and one sample was taken from a nontrafficked area (Figure 20). The cores were numbered 1 through 6, with cores 4 and 6 from the outside wheel path (closest to the shoulder of the pavement), cores 1 and 5 from between the wheel paths, core 3 from the inside wheel path, and core 2 from the nontrafficked area. The nontrafficked area was in a traffic island formed between the northbound and southbound traffic lanes. Visual examination, density determination, and extraction tests were used to evaluate the cores and to attempt to determine the cause of the bleeding. Four of the cores (numbers 1, 2, 3, and 5) were evaluated for aggregate, binder, and mixture properties. Cores 4 and 6 were used to obtain the

theoretical maximum specific gravity. These properties were then compared to values obtained during the construction of the SMA pavement.

There was visual evidence of excess asphalt cement on the surface of all of the cores except for core number 2. Core number 2 was from nontrafficked SMA placed on the second day of paving. The remainder of the cores were from the SMA placed on the first day of paving. The cores were taken full depth through the entire asphalt concrete structure to the base course material. The total pavement thickness averaged about 165.1 mm (6.5 in.). There were four discernable layers of asphalt concrete, including the SMA. The gradation, asphalt content, and asphalt cement test results of the core samples numbers 1, 2, 3, and 5 taken in 1994 are given in Table 27. Core number 2 appeared to have a somewhat harder asphalt cement, while also having the lowest asphalt content (Table 27).

The results of testing during construction on the SMA mixture are given in Table 26. These results show a greater variation between test samples than obtained between the 1994 core samples. The similarity was somewhat expected because the 1994 core samples were all taken within approximately 15.2 m (50 ft) of each other and the samples taken during construction were obtained randomly throughout the day. The average gradation of the 1994 cores versus the samples taken during construction was noticeably finer in minus 4.75 mm (No. 4) sieve material (Table 28). Also, the asphalt content of the 1994 cores was 0.35 percent higher than the average obtained during construction. The density of the laboratory samples compacted during construction were relatively consistent. There was however, a great variation between the maximum theoretical density determined by calculated methods using the virgin aggregates and those determined using ASTM D 2041.

Summary

The asphalt cement that has appeared in the wheel paths at Edwards AFB indicates that there was an excessive amount of asphalt cement in the SMA mixture. The excessive amount of asphalt cement occurred despite test results indicating that the specified requirements had been met. The gradations of the core samples obtained 1 year after construction were consistently finer below the 4.75 mm (No. 4) sieve than was indicated from test results obtained during construction. The increase in minus 4.75 mm (No. 4) material could indicate a variation not detected during construction or a breakdown of the aggregate with time. The asphalt content of the 1994 core samples was somewhat higher than that determined during construction. The combination of a mixture with a higher fine content and a higher asphalt content than was specified would contribute to the bleeding observed in the wheel paths. There was only a slight increase in field density of the SMA after 1 year of trafficking and to date no evidence of rutting.

Testing during construction of the SMA in 1993 indicated a large difference in the values obtained for the theoretical maximum specific gravity between

calculated and ASTM D 2041 methods. It has been standard practice for the Corps of Engineers on nonrecycled mixtures to use calculated methods to determine these values. Experience with the 1993 construction and information available from other sources such as the NAPA (NAPA 1994), it would appear that ASTM D 2041 should be used to determine mixture void content values. The void content should remain within a range of 3 to 4 percent.

Current guidelines on SMA, published by the NAPA (NAPA 1994), show a substantial change in the recommended gradation of SMA compared to previously available information. The major difference was in the 4.75 mm (No. 4) sieve size where the current recommendation is to specify 20 to 28 percent passing. The specification used on Lancaster Boulevard, based on European and initial projects in the U.S., had specified 25 to 35 percent passing. This change in gradation would increase the void space within the aggregate matrix of the mixture. This change was intended to decrease the possibility of rutting or bleeding of the SMA surface through movement of the mixture. Incorporating the information obtained through the demonstration project and other sources should eliminate future problems with bleeding.

Royal Air Force (RAF) Lakenheath, United Kingdom

The demonstration project at RAF Lakenheath, United Kingdom was placed on a taxiway at the air base (Photo 18). The SMA was placed on the southern side of the centerline of the southern taxiway between standing 1709 and 1710, see Figure 21. A test section of SMA was also placed on a taxiway to a hardstand at the air base. The demonstration project and the test section of SMA pavement were place in the beginning of August, 1993.

The area of SMA placed was approximately 30 m (98 ft) long and 3.7 m (12 ft) wide. The SMA's compacted thickness was approximately 40 mm (1.6 in.).

Specifications

British Standards were used to develop the specifications and construction methods used for the SMA demonstration project. Tables 29 and 30 provide a listing of mixture requirements specified for the SMA. The mixture requirements for this SMA demonstration project were similar to those used at Edwards AFB. The main differences were in compacting with a 60 versus 50-blow compactive effort and the use of a harder asphalt. The harder asphalt was used despite a much cooler climate than at Edwards AFB.

Mix design

The mix design for the SMA was developed by the local contractor who placed the SMA pavement, see Tables 29 and 30. An adaptation of the

Marshall mix design procedure using 60 blows per side was used to make the laboratory samples.

A granite was used for the coarse aggregate and a flint gravel was used for the fine aggregate. A ground limestone filler was used along with the preceding aggregates to develop the required gradation. A cellulose fiber was also added to the mixture at a rate of 0.3 percent by weight of total mixture. The fiber along with the limestone filler acted as a stabilizing additive to the SMA mixture. A 50 penetration grade asphalt cement was used for the SMA mixture at an optimum content of 6.8 percent by weight of mixture. Table 29 contains a listing of all the properties of the job-mix formula developed for the SMA.

Construction

Plant. The SMA mixture placed for the demonstration project was produced in a batch plant. There were no adaptations to the plant required for making the SMA mixture. To achieve proper mixing of the cellulose fibers within the SMA mixture, the fibers were added to the pugmill as the aggregates were added. The total dry mix time of the aggregates and fibers was approximately 30 seconds from the start of the process until the asphalt cement was added. Shorter dry mix times were investigated but they resulted in unsatisfactory mixtures.

Test section. Prior to placing the SMA for the demonstration project a small test section was placed on a nearby taxiway, see Figure 21. The test section placed was approximately 20 m (60 ft) by 3.1 m (10 ft) in area and the thickness was 40 mm (1.6 in.). The existing asphalt concrete was removed from the pavement by cold milling to the depth given above. The remaining pavement on each end where the milling started and stopped was cut vertically with a saw and removed with a pneumatic hammer.

After cleaning the area by brooming, a tack coat of K1-40 emulsified asphalt cement was applied. The SMA was placed with a conventional asphalt concrete paver. Table 29 contains the mixture properties of the SMA that was placed at RAF Lakenheath. The placement of the SMA required no special procedures. Overall the mixture had a coarse looking texture and behaved very much like porous friction mixtures. Handwork performed around the joints and within the mat itself was conducted without any obvious signs of segregation or other problems (Photo 19). As with conventional HMA the compaction operations were started soon after mix placement. The compaction procedure consisted of breakdown rolling with a 2.7 Mg (3-ton) -tandem steel-wheeled roller (Photo 20). Additional compaction was applied with a 10.9 Mg (12 ton) -three wheeled roller. Close observation of this roller revealed that it was not marking or moving the SMA surface, indicating that the 2.7 Mg (3-ton) roller was achieving the aggregate particle to particle contact desired. The rollers were able to meet the compaction requirement for the SMA pavement of 97 percent of laboratory density. Only static steel wheeled rollers were used for compaction of the SMA because vibratory rollers would tend to

break aggregate or cause bleeding of the binder and rubber-tire rollers tend to pick up fines and asphalt cement from the binder rich mixture.

Placement of SMA demonstration section

To place the SMA mixture within this area, approximately 40 mm (1.6 in.) of asphalt concrete was removed by cold milling, see Photo 18. The area was then cleaned and tack coated as described above for the test section. The SMA was placed in this area at the same thickness of the asphalt concrete that was removed.

The SMA mixture placed in the demonstration project was similar to that used for the test section. The SMA pavement was placed with a paver and compacted with the same equipment and procedures that were used to construct the test section. The SMA mixture was trucked from the asphalt plant to the construction site over a distance of 56 km (35 miles).

The perimeter of the completed SMA pavement was overband sealed with a hot pour asphalt sealant. Three core samples were taken from the completed SMA pavement for density and thickness determinations.

Testing

Visual observation of the core samples taken after construction showed good aggregate distribution had been achieved and measurements indicated that compaction requirements had been met. Tests for surface smoothness using a 3.7 m (12 ft) straightedge revealed that the SMA pavements did not exceed the maximum allowable deviation from the straightedge of 3 mm (1/8 in.).

Summary

The performance of the SMA placed for the demonstration was reported by local personnel to have been very good. Two years after placement there were no signs of distress to the SMA. However, the subject test section of SMA was removed within 2 years as part of a reconstuction project for the entire taxiway. The gradation used for this demonstration was very similar to the one used at Edwards AFB; however, the asphalt cement used for this demonstration was a harder grade of asphalt cement. Also compared to Edwards AFB this SMA did not receive as much traffic.

5 Conclusions

The objective of this study was to evaluate and compare SMA mixture properties to a standard airfield HMA (control mixture). This was accomplished through a literature review and site inspections of several SMA road construction projects, and a laboratory evaluation program. The laboratory evaluation included the Marshall mix design method, indirect tensile, and creep-rebound testing was used to obtain data for comparison. In addition two field demonstration projects were constructed to identify possible areas of concern with SMA construction. The following conclusions were developed from this study.

Literature Search

The 50-blow Marshall procedure is the most common method of specimen compaction for SMA design. The voids in the total mixture should be about 3 percent. Marshall stability and flow values are not critical parameters in the selection of the proper asphalt content or the satisfactoriness of the mixture. Compared with dense graded mixtures of the same materials the stability values of SMA mixtures are generally 50 to 60 percent lower and flow values 14 to 70 percent higher with the same materials in a dense graded mixture.

Production capabilities with SMA maybe somewhat lowered when compared to conventional mixtures due to increased dry and wet mixing times. Recent work by U.S. researchers indicates that the gradation should be coarser than those normally used in Europe. The coarse aggregate should be 100 percent crushed. The coarse aggregate, exceeding the 4.75 mm (No. 4) sieve size, should make up approximately 80 percent of the total aggregates.

The Europeans generally use a harder grade of asphalt compared to SMA's placed in the U.S. The particle size of the material passing the 150 μm (No. 200) sieve can have an effect on the asphalt content. Generally, the coarser this material is the higher the required asphalt content.

Cellulose fibers in the range of 0.3 percent by weight of the SMA are very effective in preventing asphalt drainage during construction. Modified asphalts have proven successful at controlling excessive drainage and also increasing the

low temperature properties. The drainage test can be a satisfactory method of determining the suitability of the developed asphalt content.

Despite poor laboratory performance by SMA mixtures in typical laboratory tests, such as, stability, flow, indirect tension, and resilient modulus, when compared to conventional dense graded mixtures; does not perform as well. Field experience has proven that SMA mixtures provide excellent performance.

Laboratory Evaluation

The optimum asphalt content for SMA mixtures should be approximately 1 percent higher than conventional dense graded mixtures constructed with the same aggregates. The retained stability of an SMA mixture should be greater than or equal to the value obtained with a dense graded mixture constructed with the same aggregates.

Marshall method test values for stability are not a good indicator of overall satisfactoriness of the SMA mixture. The stability values will generally be appreciably below the 8,010 N (1,800 lb) minimum used for airfield mixtures.

There was a notable decrease in tensile strength with an increase in temperature for both the control and the SMA mixtures. The deflection at maximum load generally increases with increasing temperature for both mixtures. For the control mix there seemed to be a peak in deflection at maximum load at 25 °C (77°F). The elastic modulus decreased with increasing temperatures for both mixtures.

Compared to the control mixture, the creep-rebound test indicated that the SMA mixture will creep twice as much but rebound slightly less. Similar results were obtained when comparing Marshall hand hammer compacted specimens to those compared with a GTM. Generally creep and rebound increased with increasing asphalt contents. These results do not correlate with the field performance of SMA mixtures, indicating that creep test results and Marshall properties are not indicative of the rutting potential of SMA mixtures.

Field Demonstrations

SMA mixtures can be produced and placed using conventional asphalt paving mixing and placement equipment without the need for special adaptations. The mixing dry and wet mixing times used to produce the SMA mixtures at Edwards AFB were only extended by a total of 5 sec over a conventional dense graded mixture (45 s versus 40 s).

Two to four passes with a steel-wheel roller were sufficient to achieve densities of 94 percent of theoretical maximum density on the demonstration projects.

Areas that require handwork will generally be more visually obvious than with conventional mixtures due to segregation of the SMA because of the coarseness of the gradation.

6 Recommendations

Based on the conclusions derived from this study, the following recommendations are made:

SMA pavement should be considered as an acceptable alternative for airfield construction or pavements in areas where rutting is a concern. The probability of foreign object damage (FOD), through loss of aggregate, should be lower than open graded mixtures (i.e., porous friction course) because of the large amount of fines that make up the asphalt matrix part of the mix.

Laboratory evaluation methods need to be developed to more adequately characterize the field performance of SMA materials.

Although not a part of this study, SMA mixtures may have application in areas where the reduction of reflective cracking is a consideration. The field performance of the SMA at Edwards AFB showed no reflective cracking after 3 years of service.

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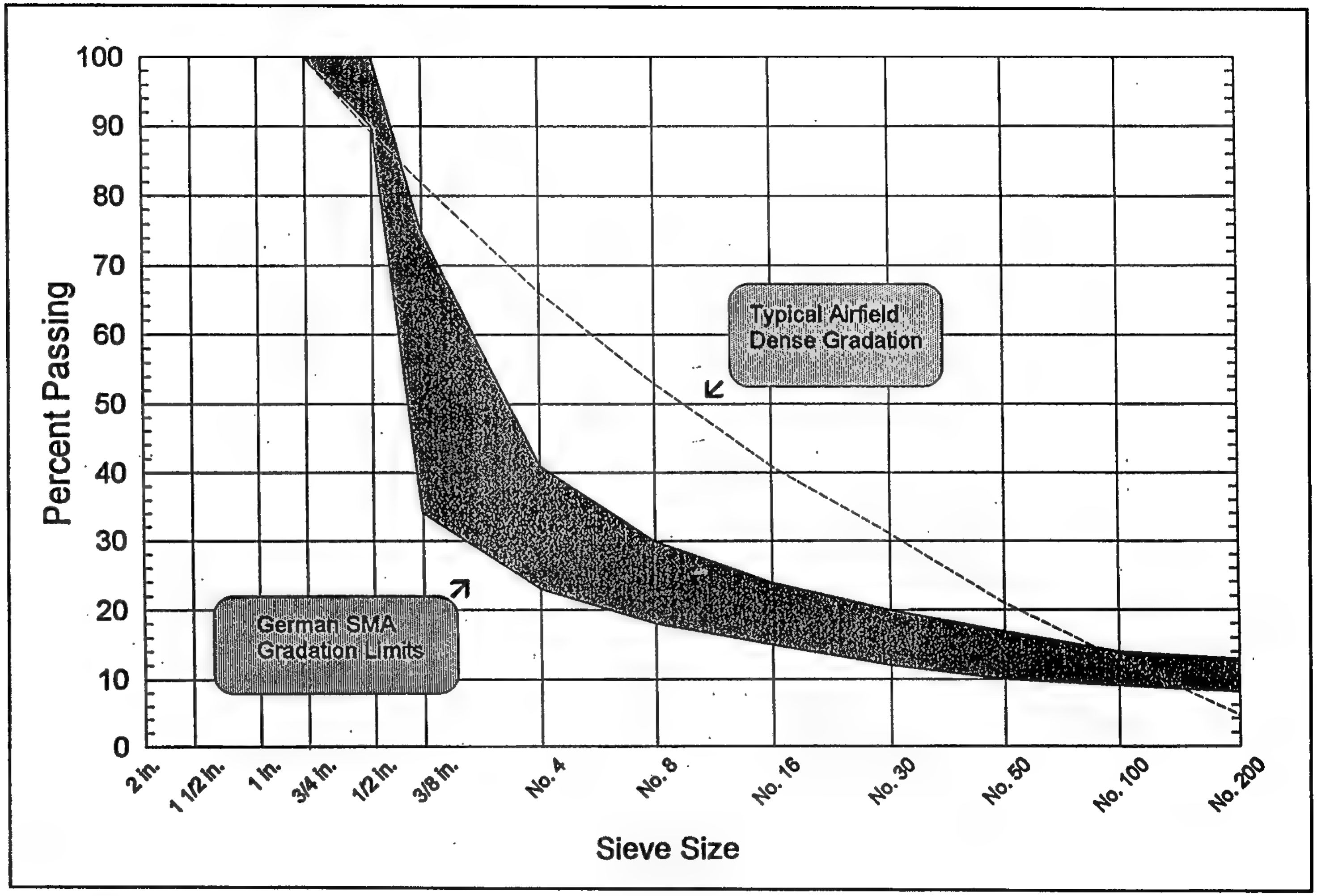


Figure 1. A comparison of SMA and typical airfield gradations

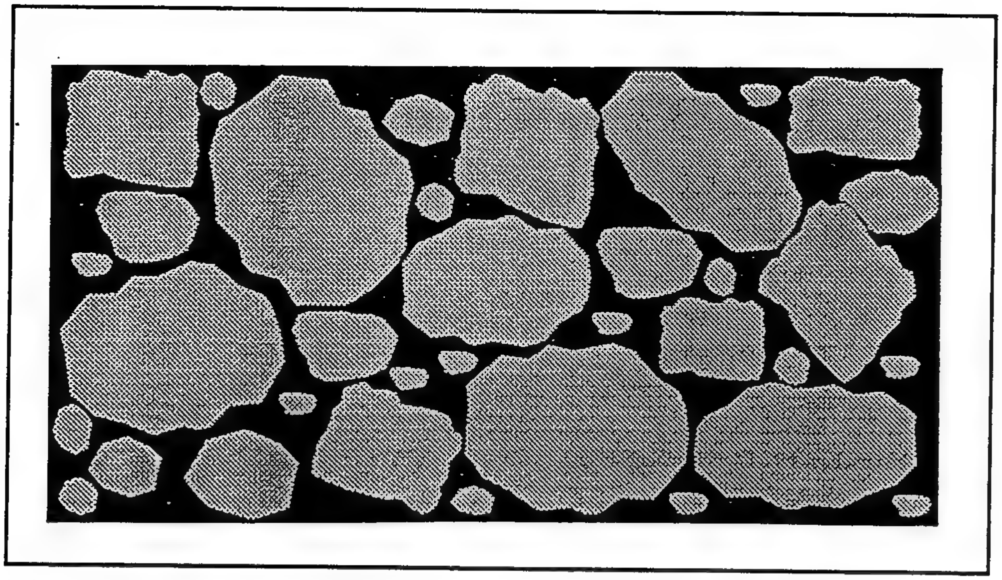


Figure 2. Schematic representation of SMA mixture

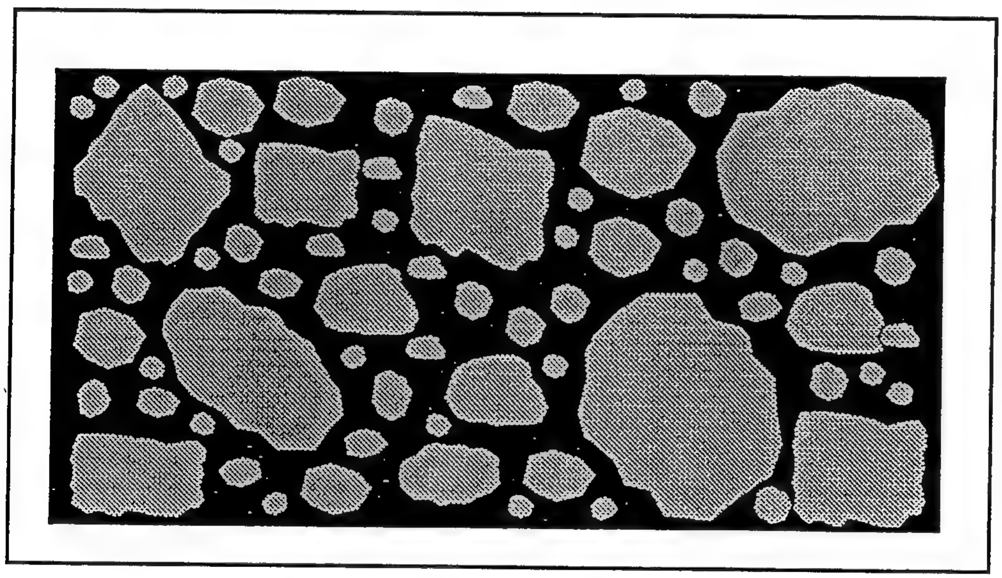


Figure 3. Schematic representation of dense-graded mixture

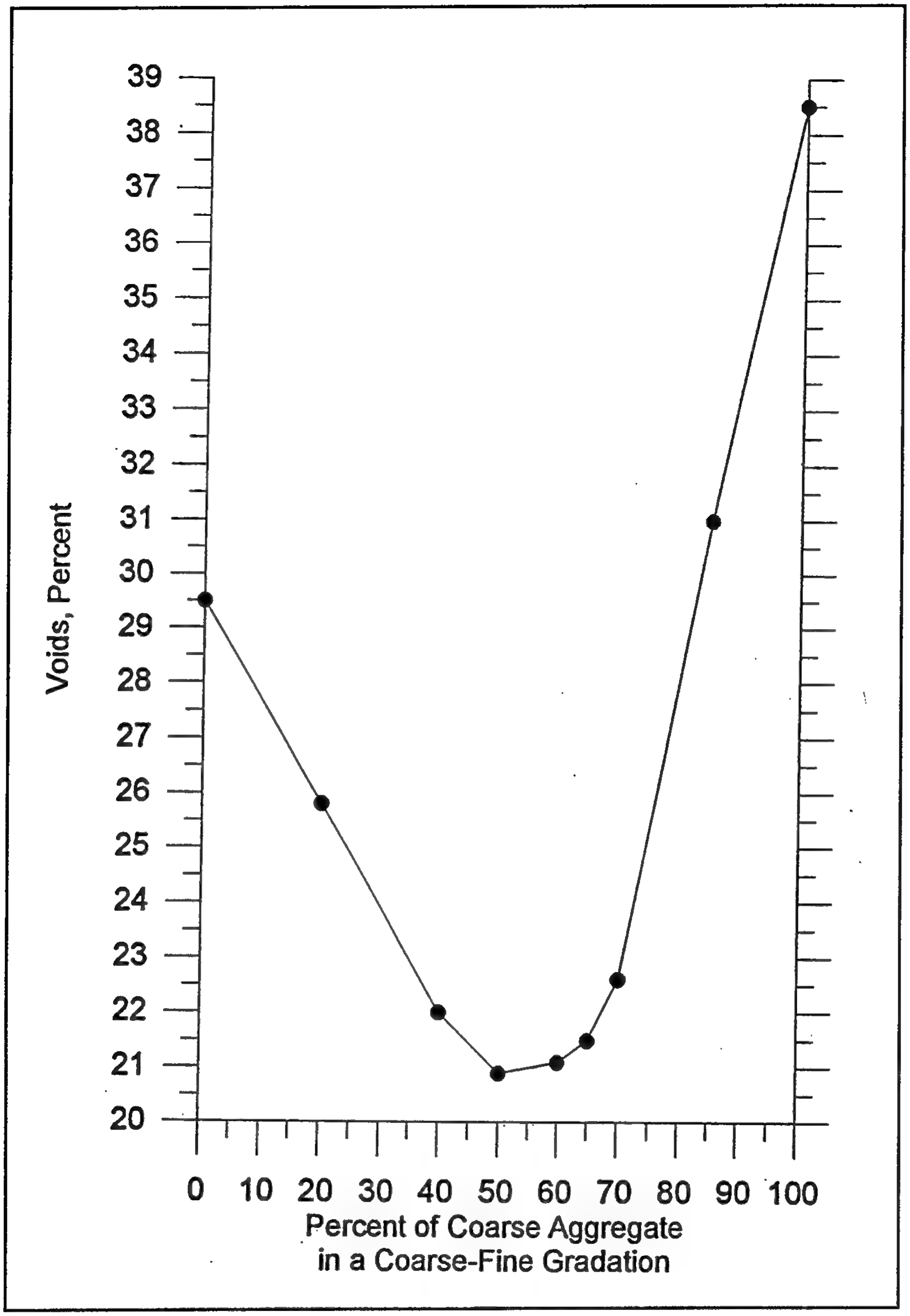


Figure 4. Relation of voids to aggregate mixture (after van der Heide 1992)

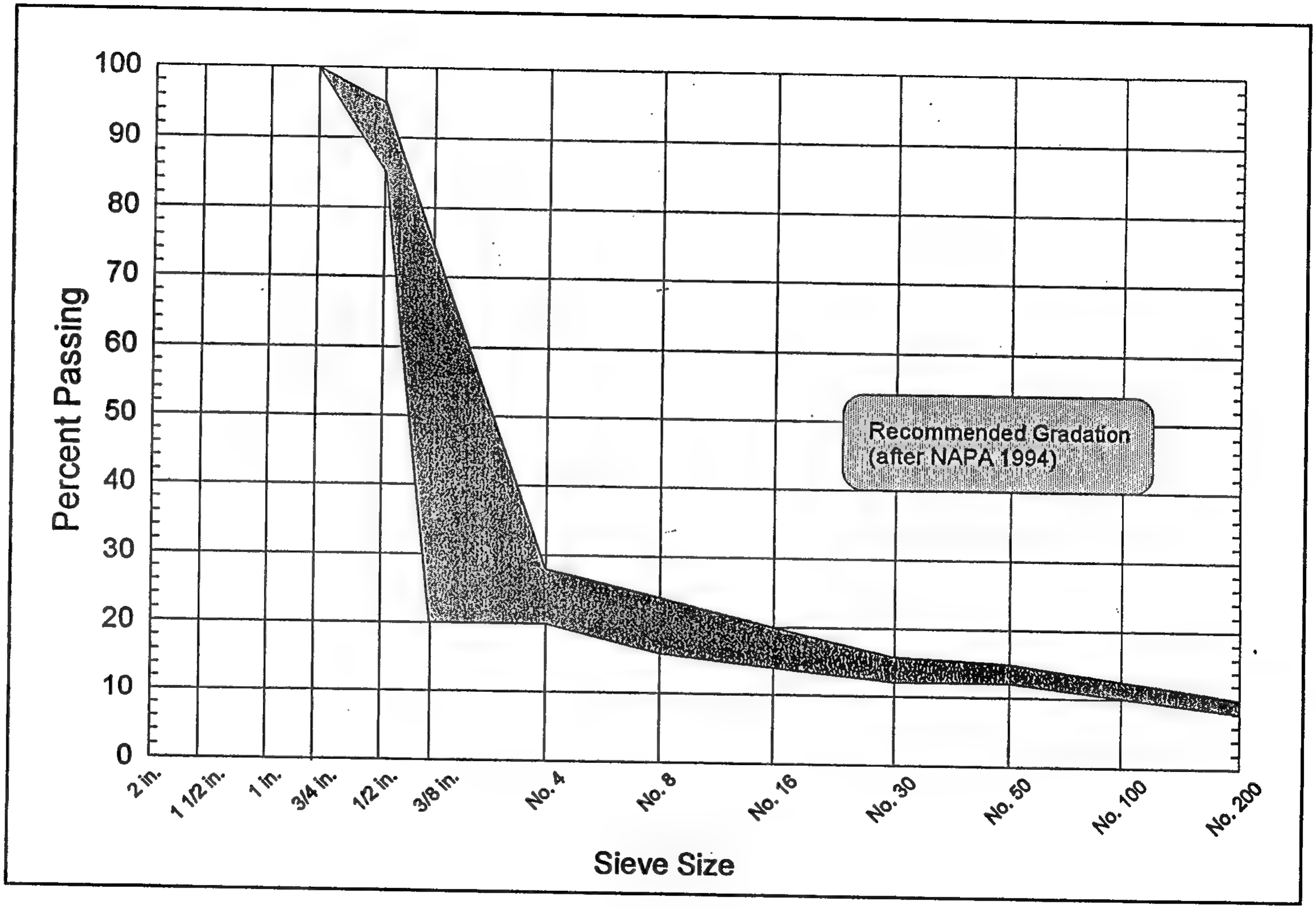


Figure 5. Recommended SMA gradation (after NAPA 1994)

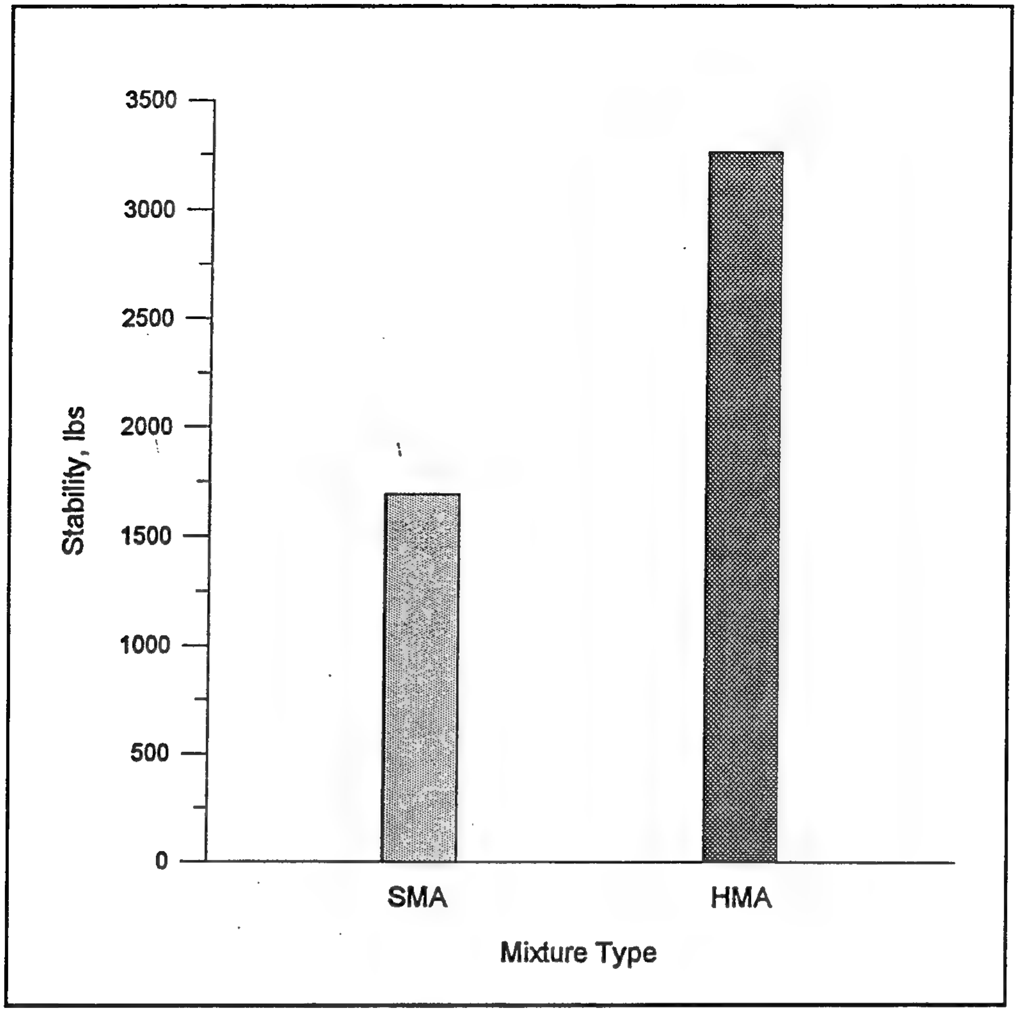


Figure 6. A comparison of SMA versus hot-mix asphalt stability values (after Brown 1992)

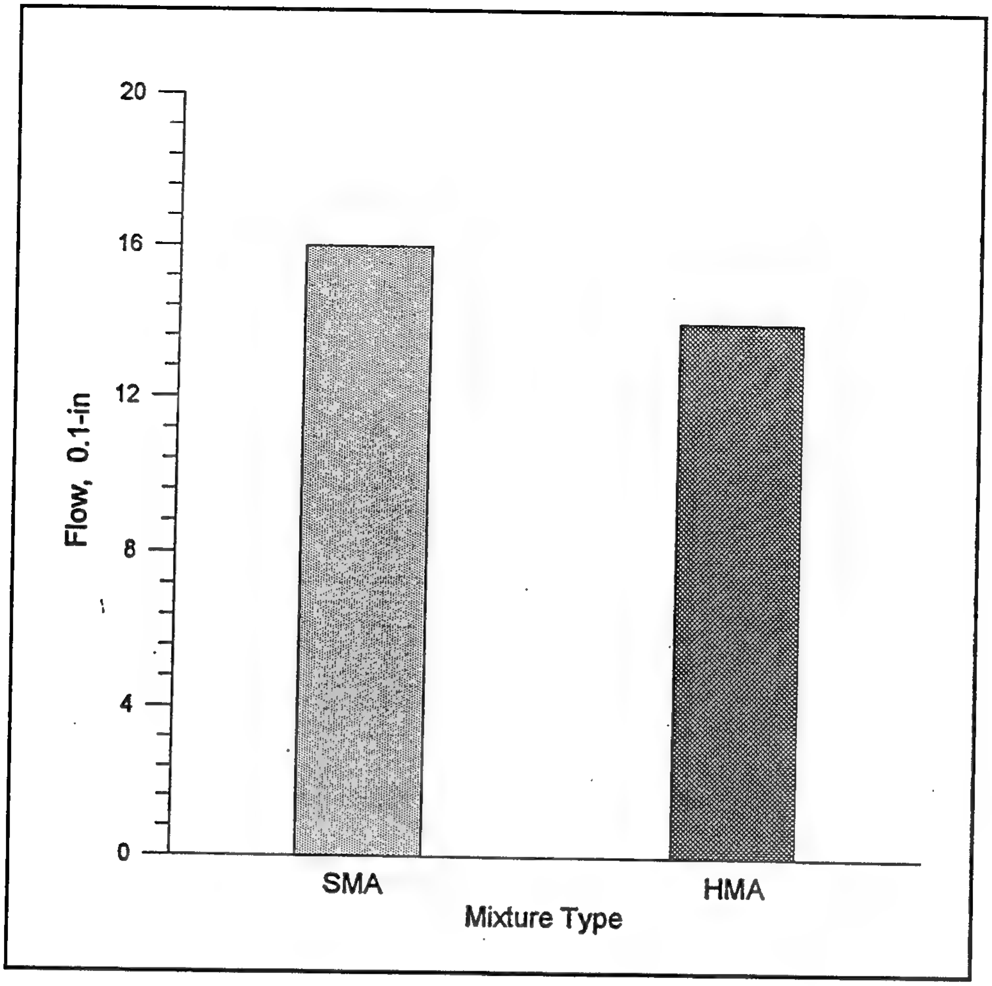


Figure 7. A comparison of SMA versus hot-mix asphalt flow values (after Brown 1992)

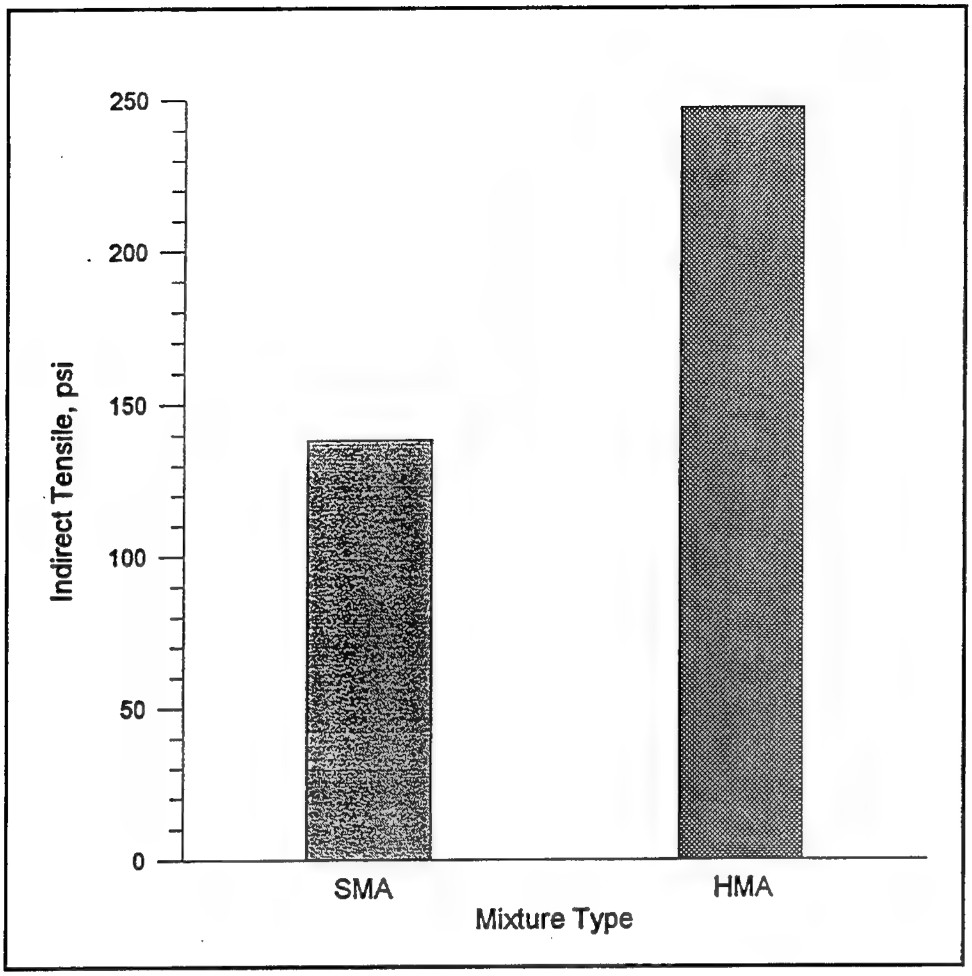


Figure 8. A comparison of SMA versus hot-mix asphalt indirect tensile values (after Brown 1992)

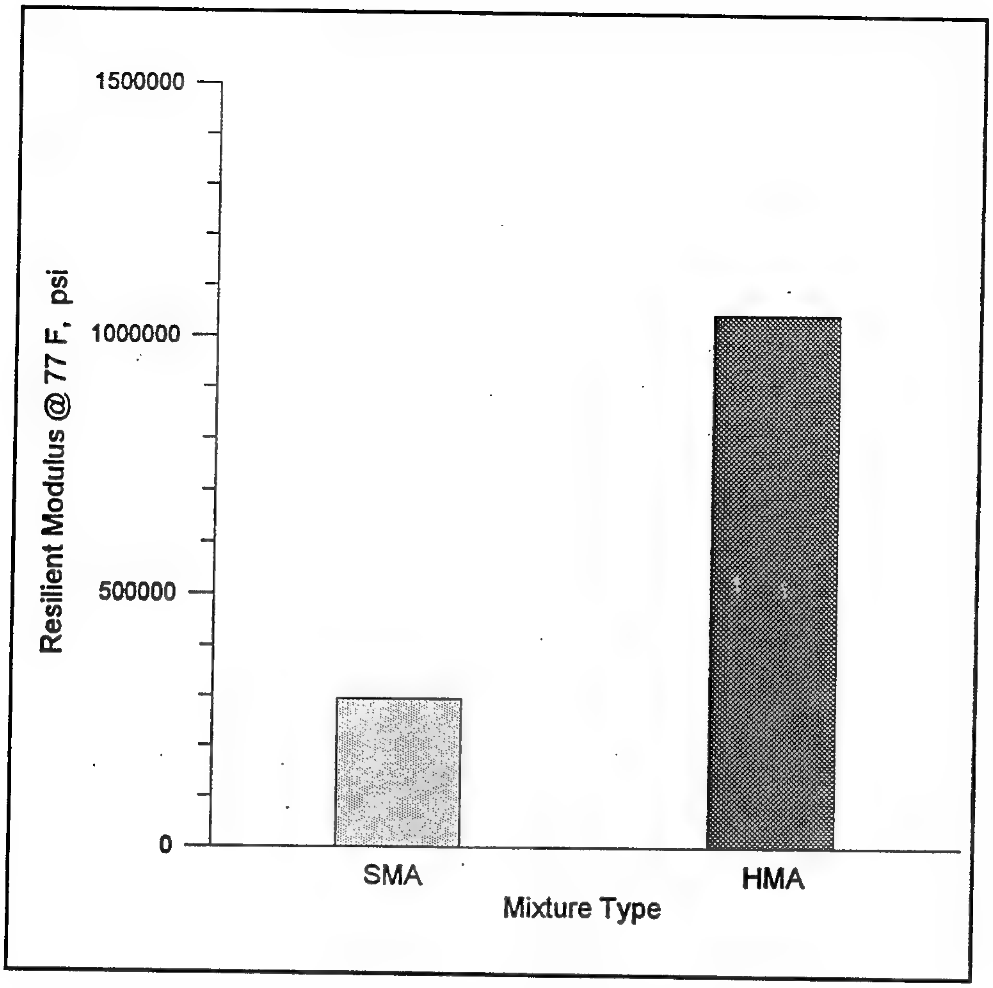


Figure 9. A comparison of SMA versus hot-mix asphalt resilient modulus values (after Brown 1992)

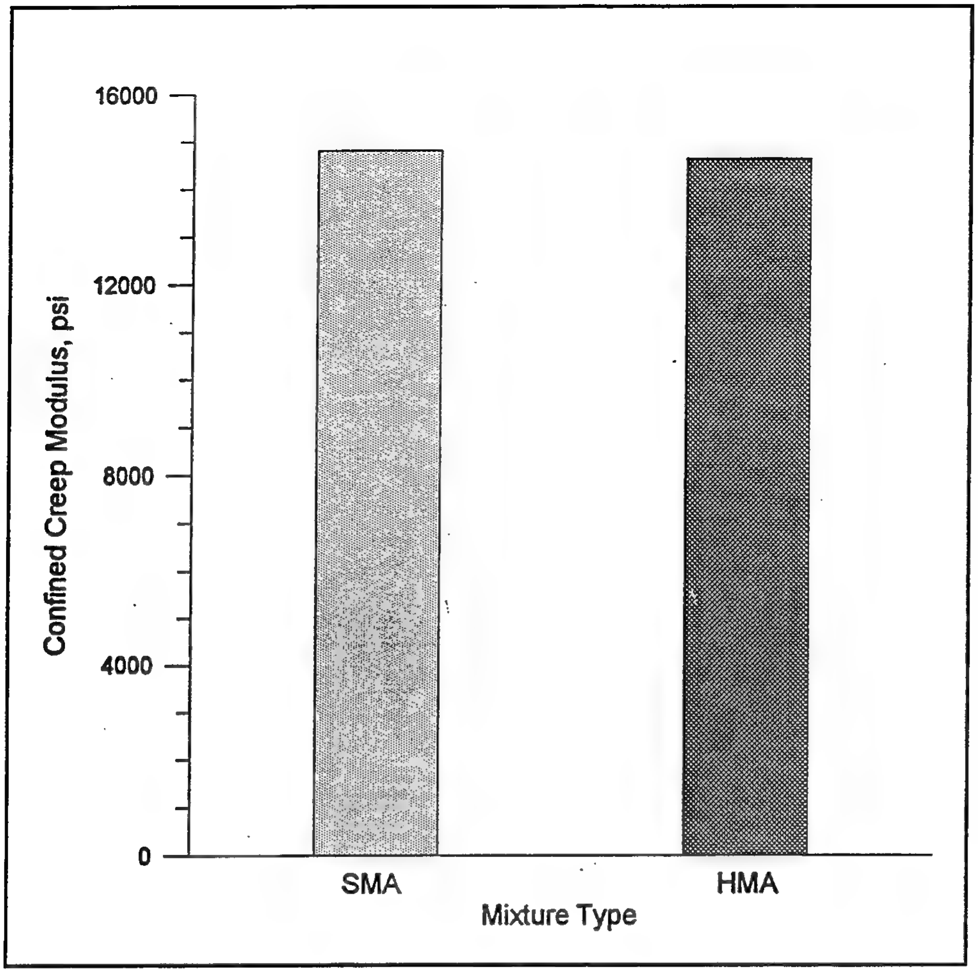


Figure 10. A comparison of SMA versus hot-mix asphalt confined-creep modulus values (after Brown 1992)

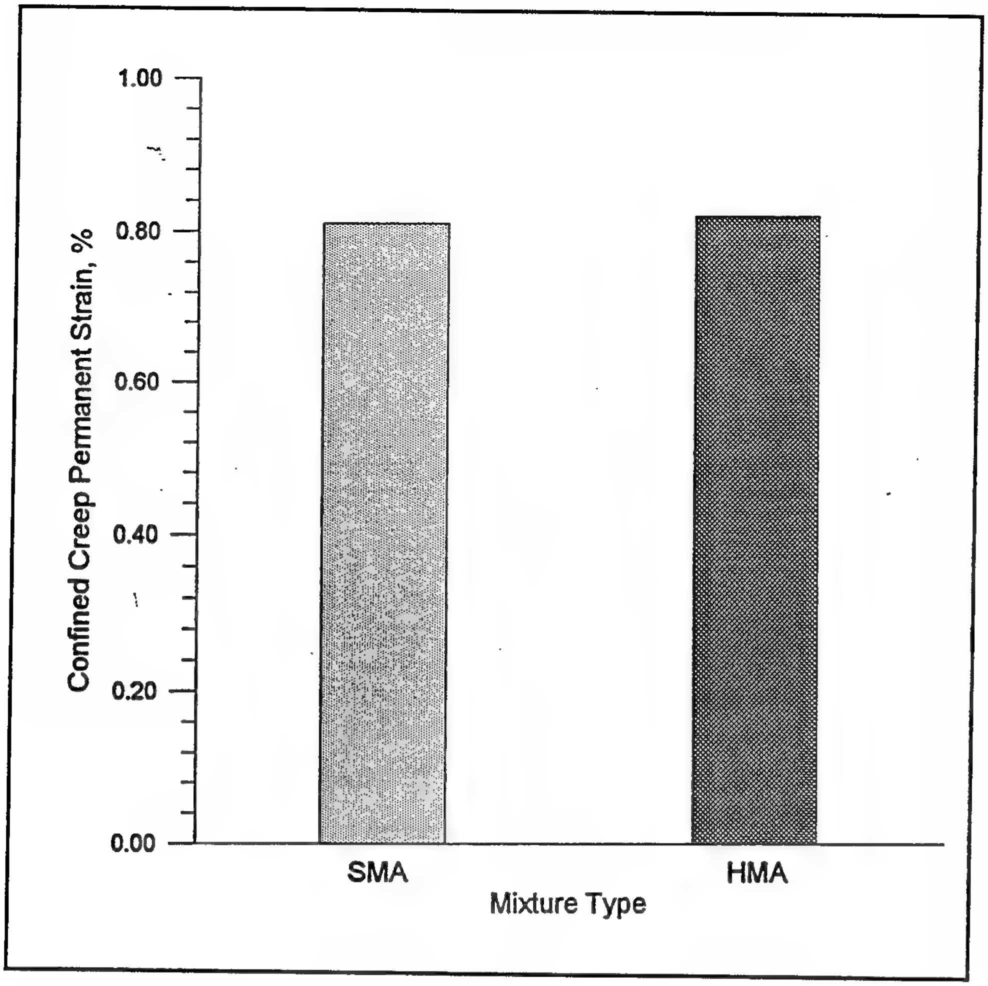


Figure 11. A comparison of SMA versus hot-mix asphalt confined-creep permanent strain values (after Brown 1992)

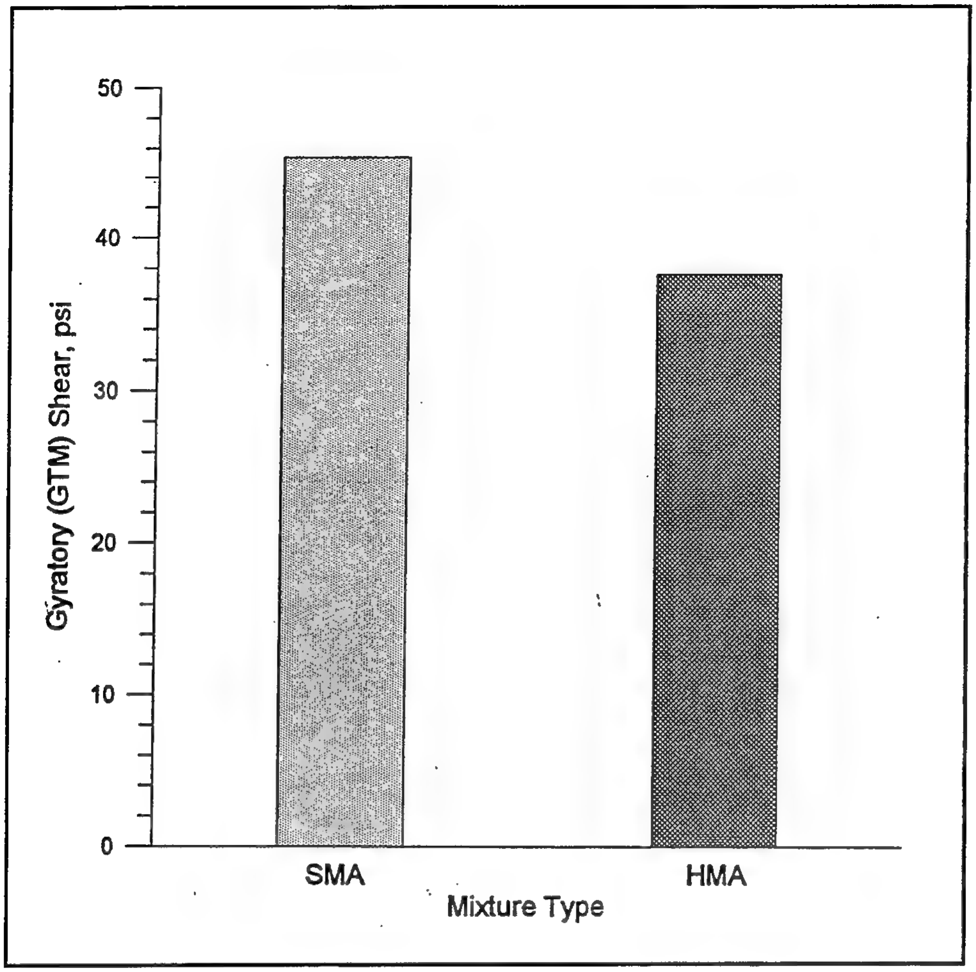


Figure 12. A comparison of SMA versus hot-mix asphalt gyratory shear values (after Brown 1992)

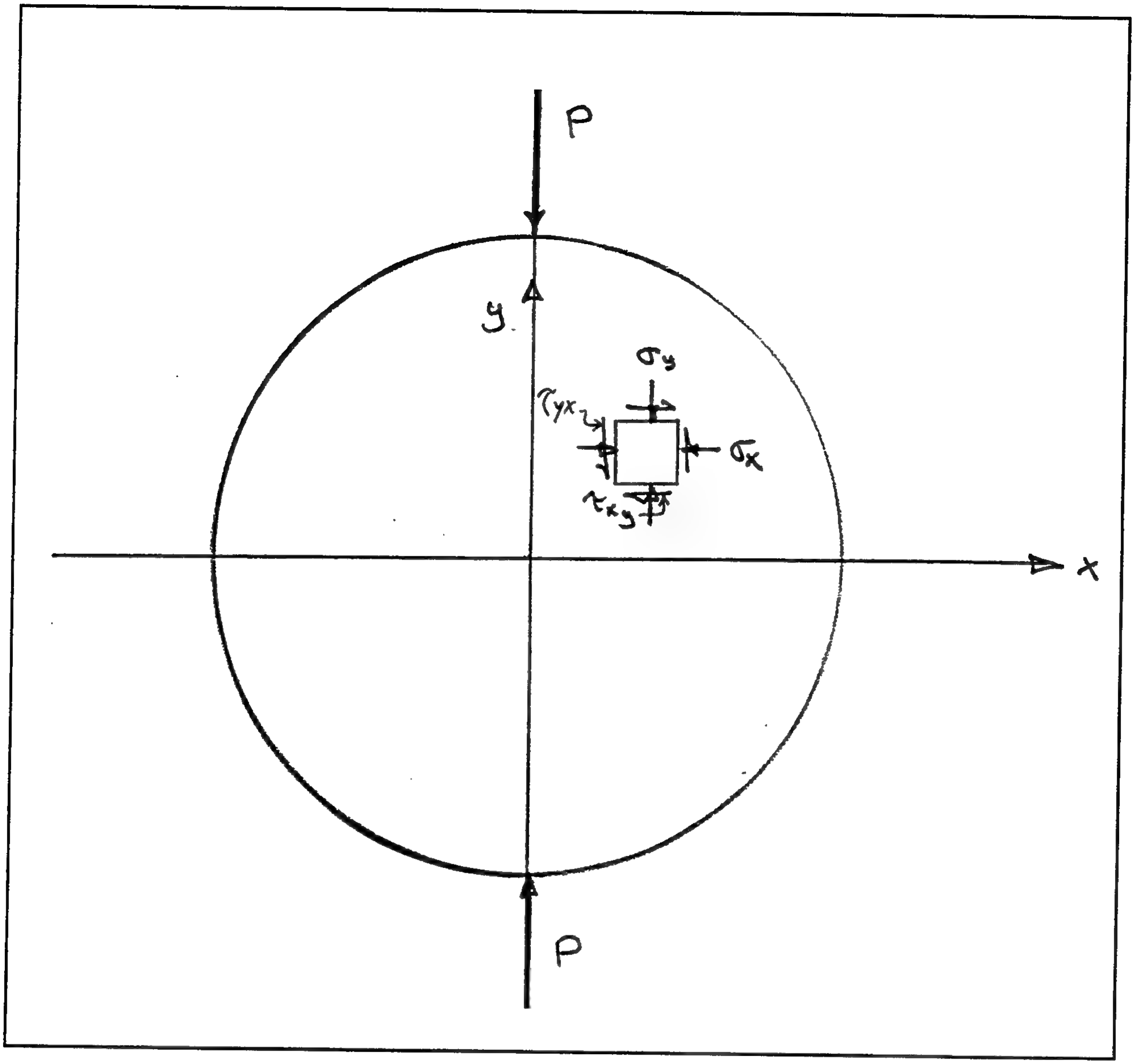
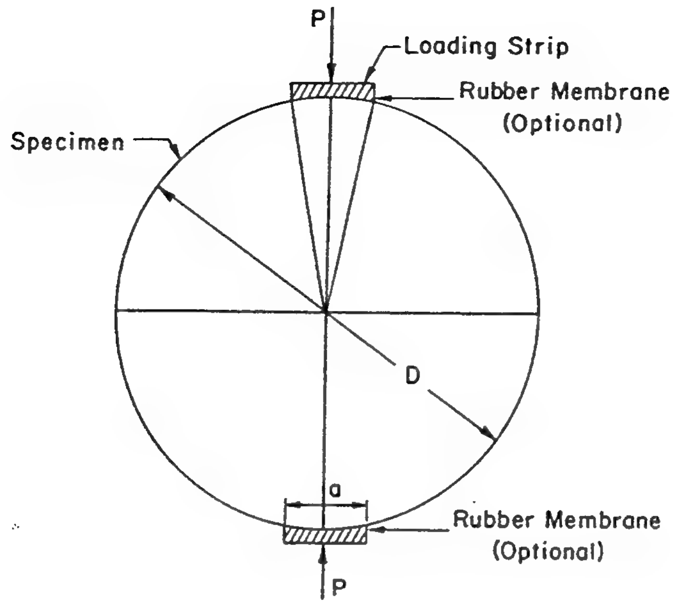


Figure 13. Schematic diagram showing stress components in a disk under the action of two diametrically opposite concentrated loads



P = applied load
t = thickness of specimen
D = diameter of specimen
a = width of loading strip

Figure 14. Indirect tension test

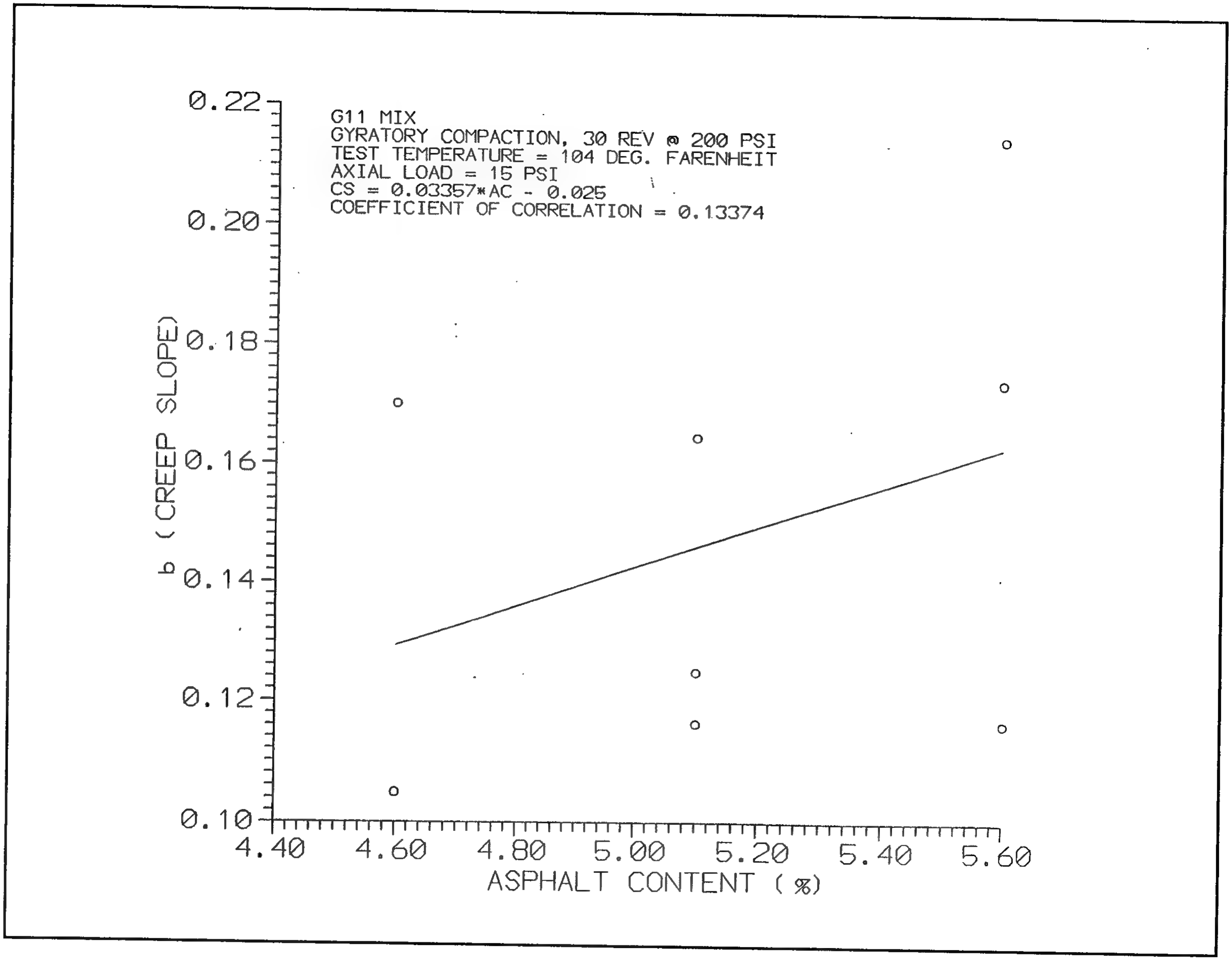


Figure 15. Logarithmic creep slope versus asphalt content for the control (GU) mixture

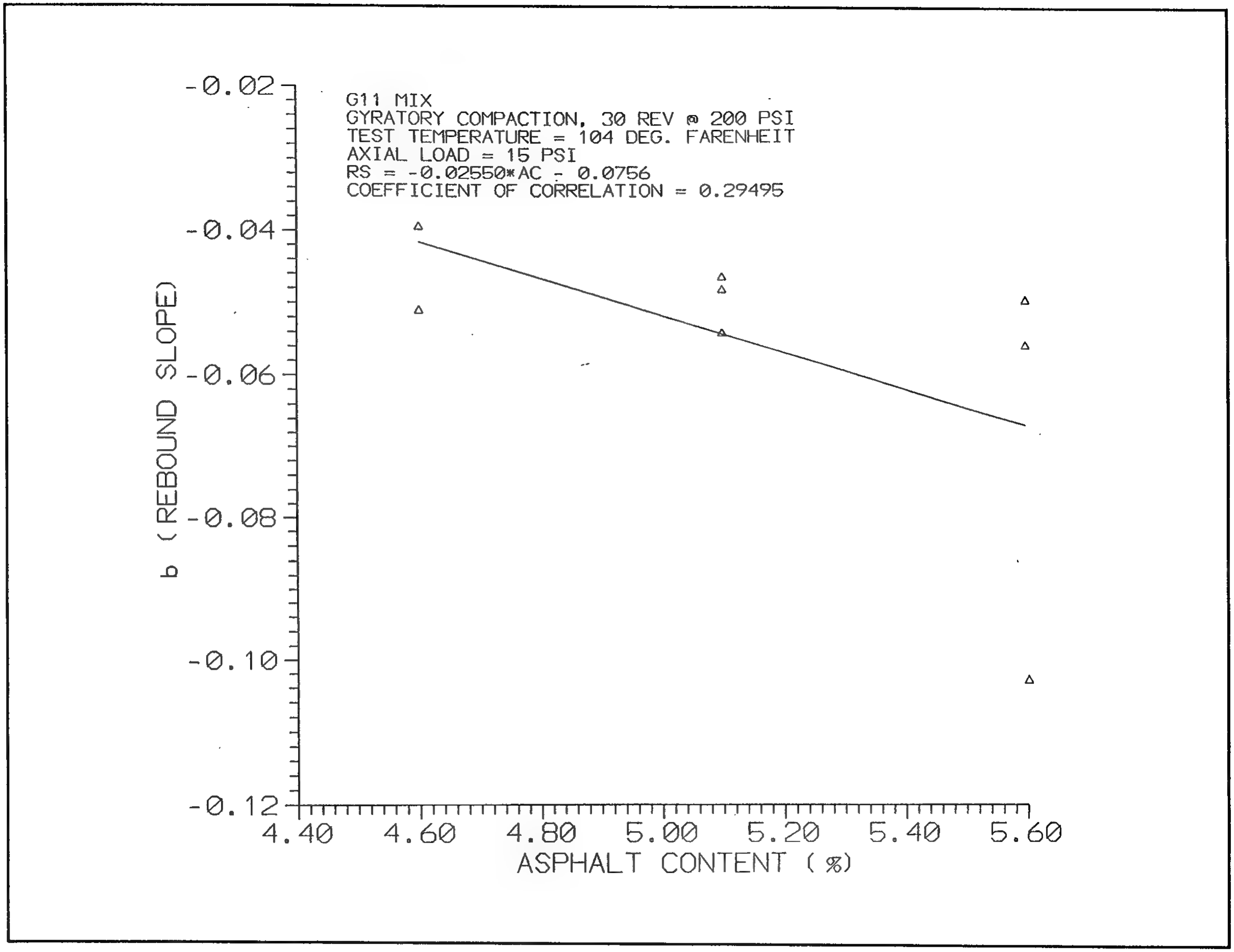


Figure 16. Logarithmic rebound slope versus asphalt content for the control (GU) mixture

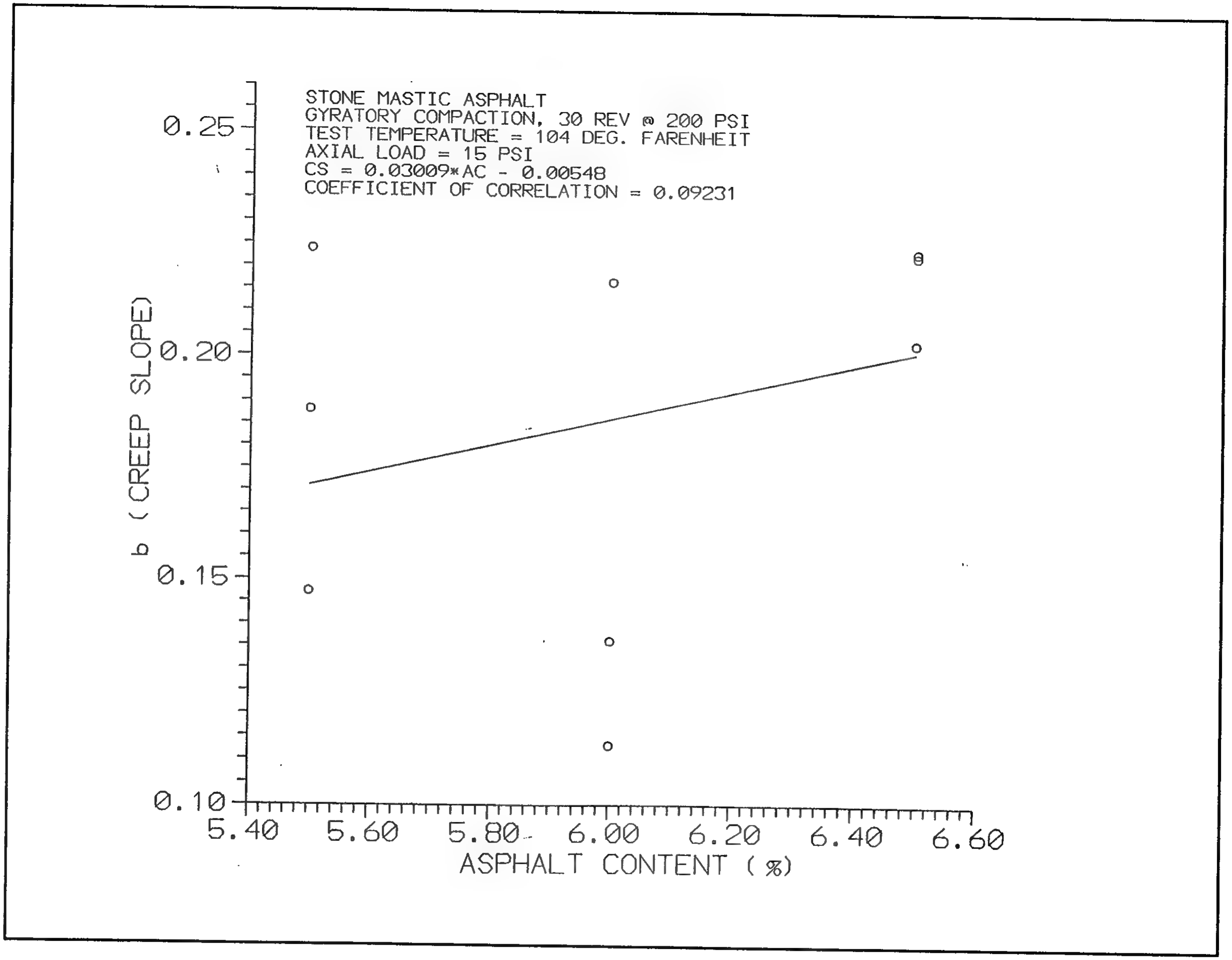


Figure 17. Logarithmic creep slope versus asphalt content for the SMA mixture

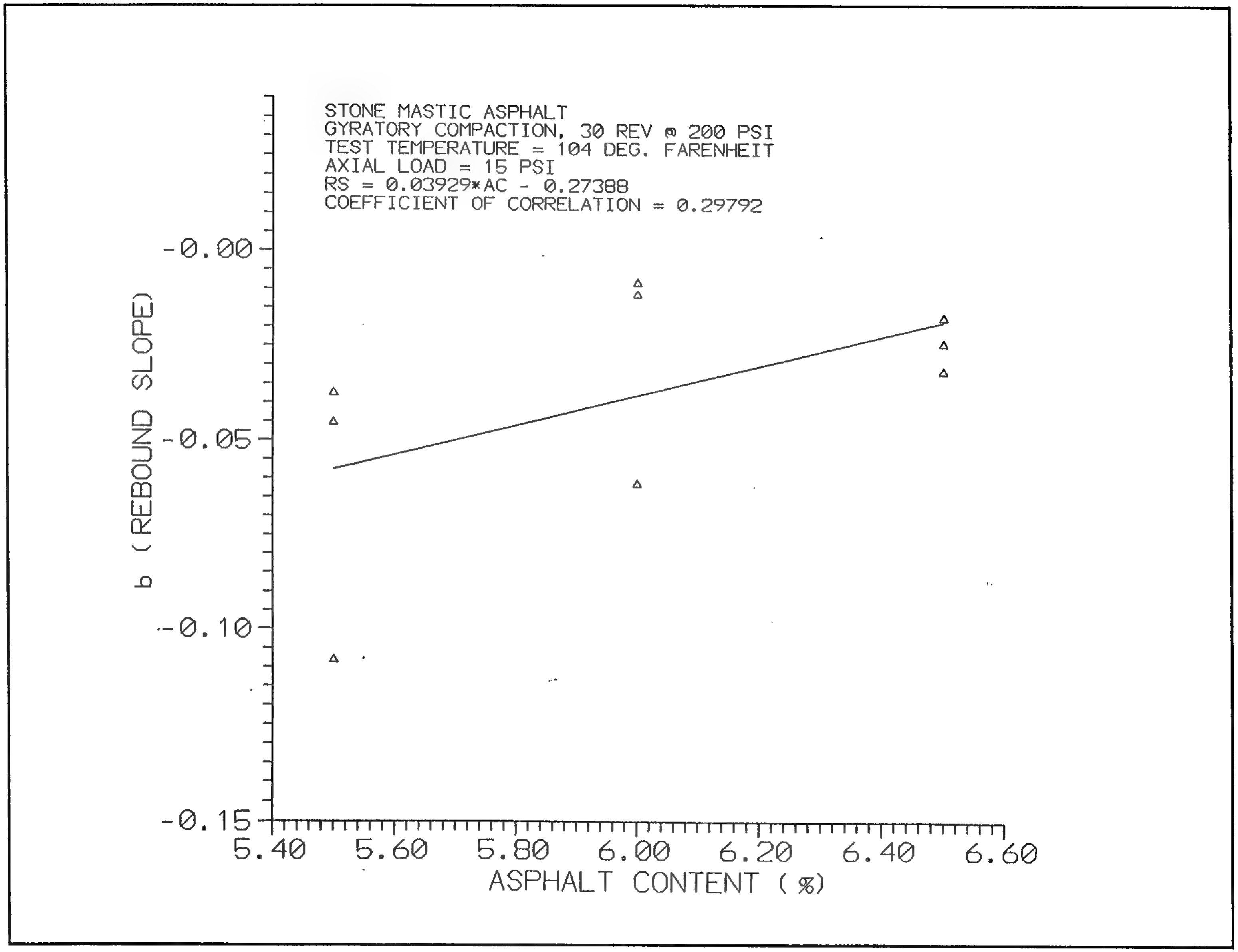


Figure 18. Logarithmic rebound slope versus asphalt content for the SMA mixture

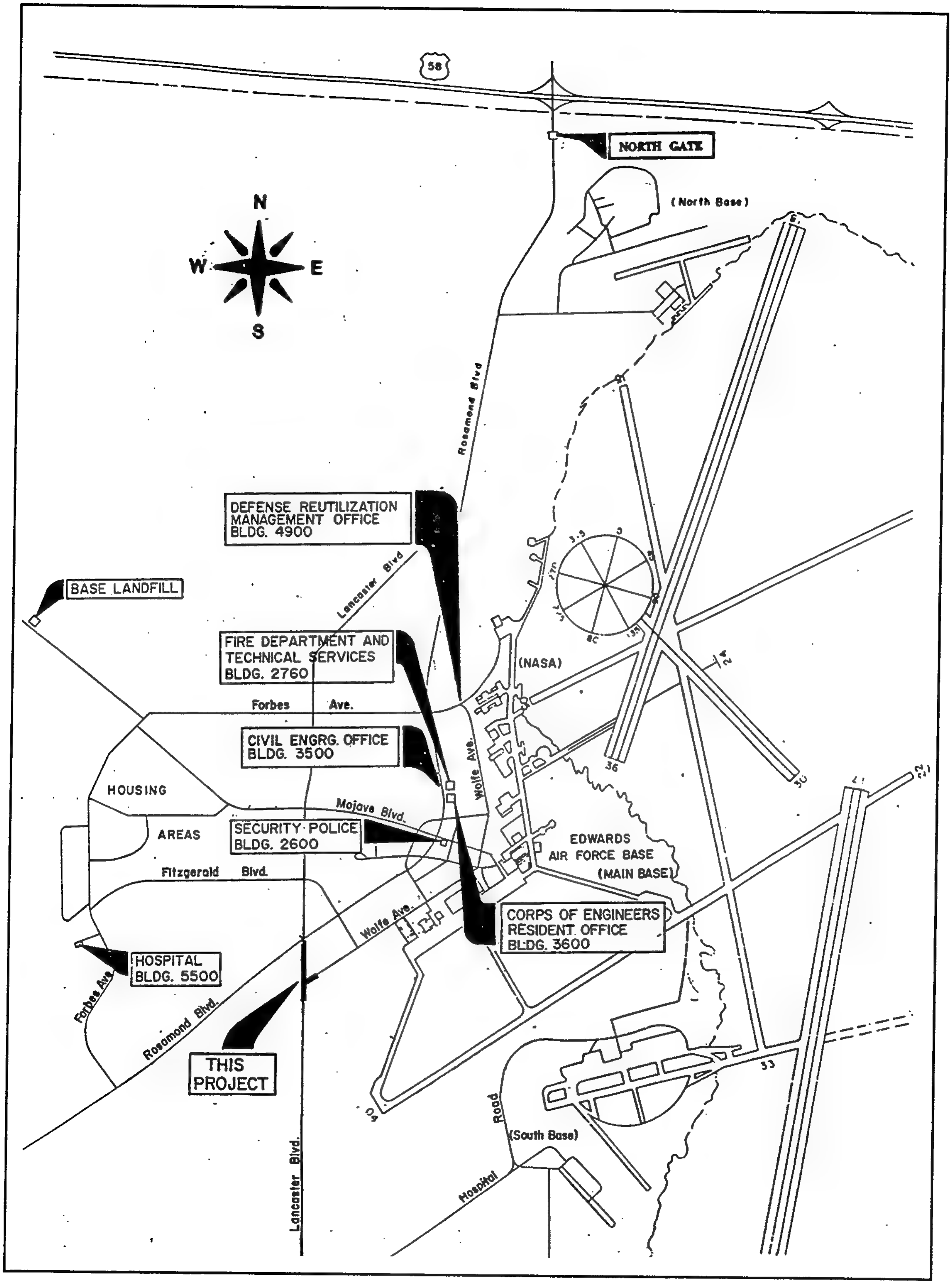


Figure 19. Location of SMA demonstration project at Edwards AFB, CA

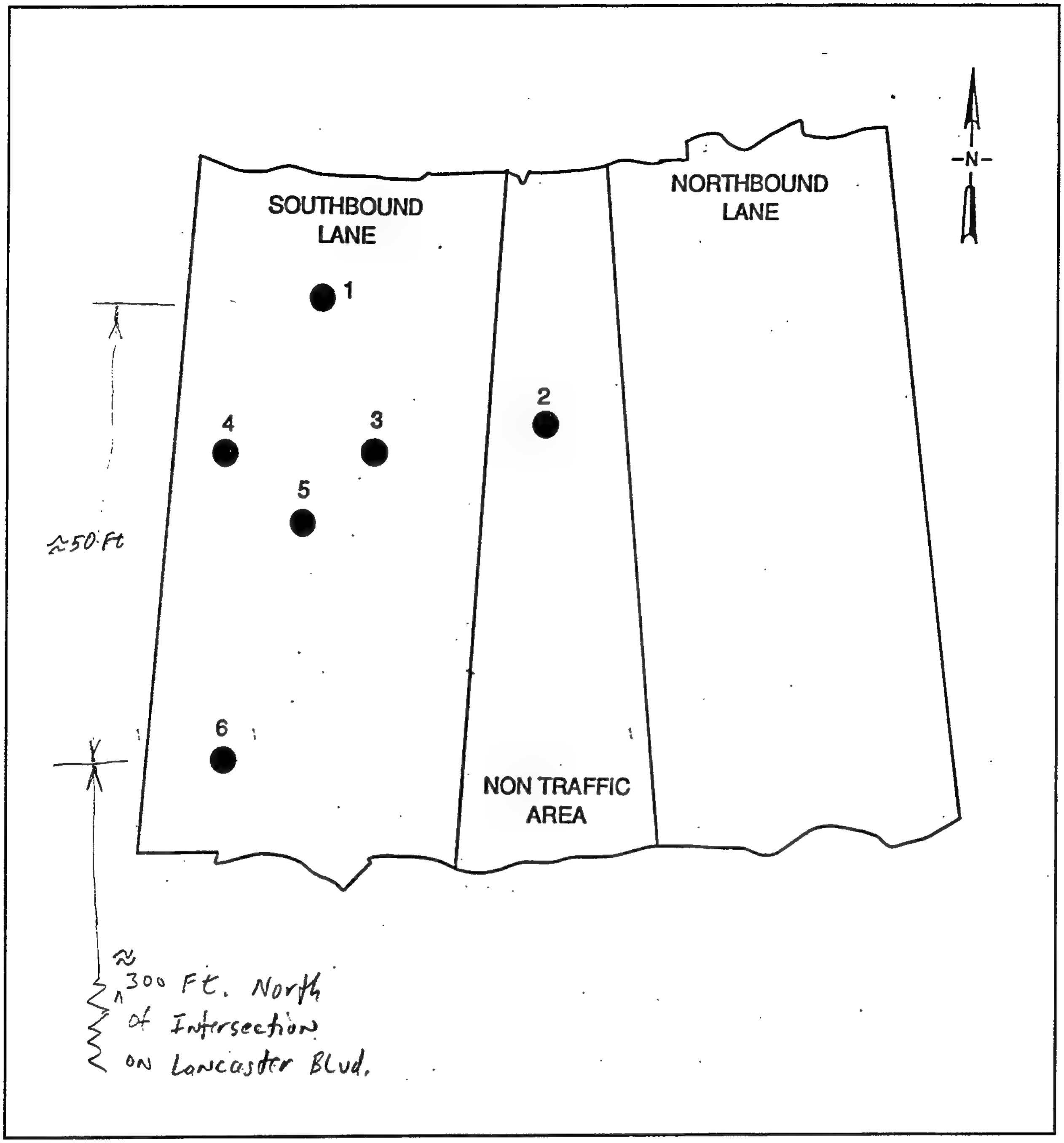


Figure 20. Location of core samples from SMA demonstration project at Edwards AFB, CA

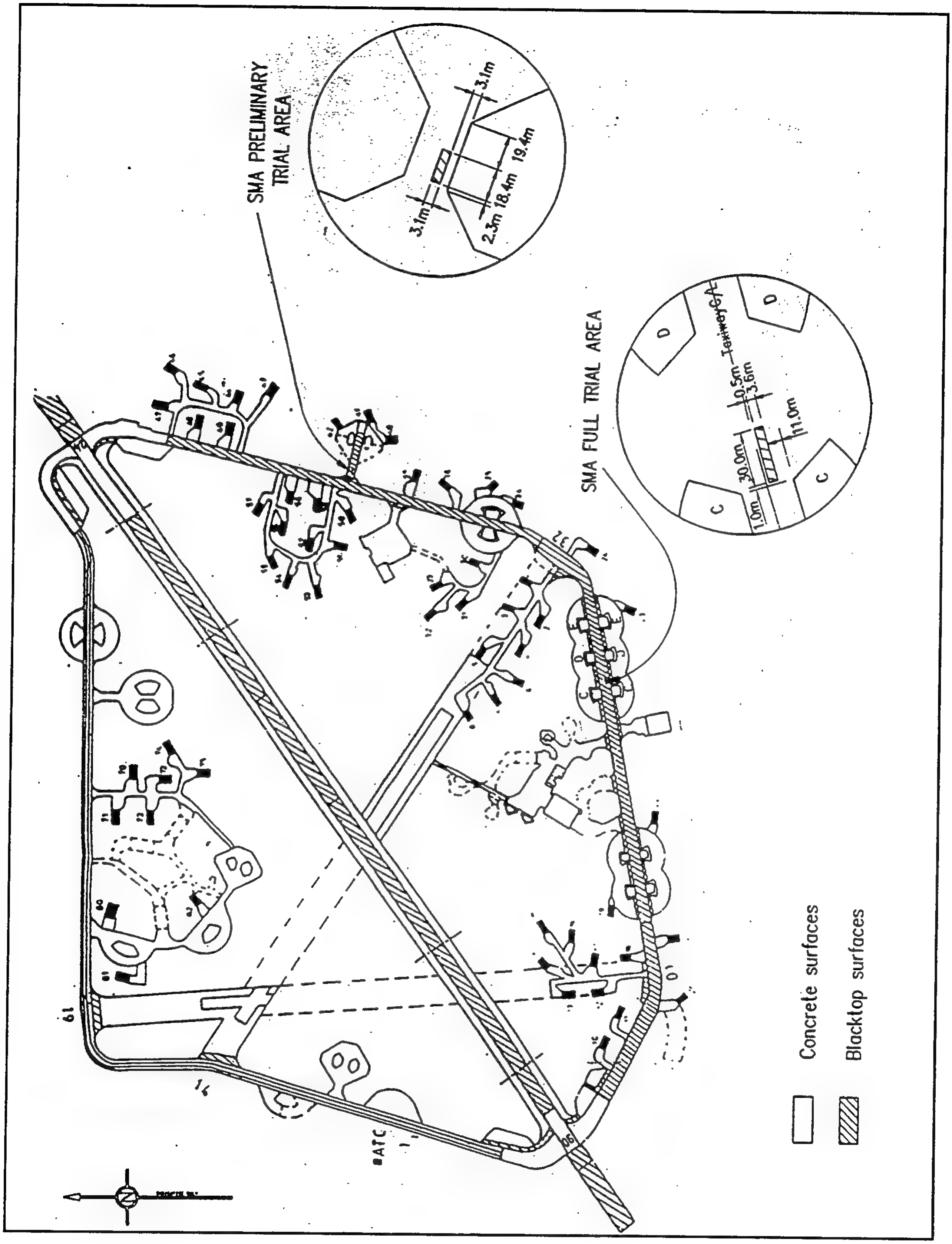


Figure 21. Layout of SMA sections at RAF Lakenheath, United Kingdom

Table 1
Typical European SMA Gradations (after Stuart 1992)

Sieve Size	Percent Passing Sieve Size				
	German Specifications			Swedish Specifications	
	19 mm	12.7 mm	9.5 mm	19 mm	12.7 mm
25.4 mm	100			100	
19 mm	90-100	100		95-100	100
12.7 mm	33-66	90-100	100	33-54	95-100
9.5 mm	26-50	34-75	90-100	26-40	34-49
4.75 mm	19-34	23-41	28-50	19-33	23-37
2.36 mm	16-26	18-30	21-34	16-29	18-30
1.18 mm	14-23	15-24	16-25	14-27	15-27
600 μm	12-20	12-20	12-20	12-24	12-24
300 μm	10-17	10-17	10-17	10-21	10-21
150 μm	9-14	9-14	9-14	9-16	9-16
75 μm	8-13	8-13	8-13	8-13	8-13

Table 2
Effect of Fines on Properties of SMA Mixtures (after Brown 1992b)

Properties	SMA				
Fines (-4.75 mm), percent	26	31	36	41	46
Asphalt content, percent	6.5	6.5	6.5	6.5	6.5
Cellulose, percent	0.3	0.3	0.3	0.3	0.3
Unit weight, kg/m^3	2,345.3	2,417.4	2,443.1	2,468.7	2,471.9
VMA[1], percent	20.3	17.7	17.0	15.8	16
VTM[2], percent	5.5	2.5	1.6	0.4	0.02
Stability, N	4,516.8	6,968.7	7,520.5	8,232.5	9,122.5
Flow, 0.25 mm	12	14	16	20	22
Indirect Tensile, at 25°C, KPa	743.3	911.5	953.5	1,061.8	992.2
Resilient Modulus, 10^3 KPa					
at 4.4°C	7,412	10,521	15,175	10,439	8,722
at 25°C	2,813	3,268	2,034	3,937	3,551
at 40°C	827	807	1,269	945	951
Gyratory (GTM)					
GSI[3]	1.0	1.0	1.0	1.11	1.34
Shear, KPa	282.7	284.1	313.0	211.7	84.1
Confined Creep					
Modulus, MPa	106.1	125.4	102.1	73.9	53.4
Permanent Strain, percent	0.78	0.66	0.81	1.12	1.55

[1] Voids in mineral aggregate (VMA).
[2] Voids in total mixture (VTM).
[3] Gyratory stability index (GSI).

Table 3
SMA Gradation Guideline (after NAPA 1994)

Sieve Size	Percent Passing
19 mm	100
12.7 mm	85-95
9.5 mm	75 Maximum
4.75 mm	20-28
2.36 mm	16-24
600 μm	12-16
300 μm	12-15
75 μm	8-10

Table 4
Recommended SMA Coarse and Fine Aggregate Properties (after NAPA 1994)

Property	Specification	Requirement
Coarse Aggregate:		
Los Angeles Abrasion, percent	AASHTO T 96	30 max.
Flat and Elongated, +4.75 mm	ASTM D 4791	
3 to 1 (length to thickness), percent		20 max.
5 to 1 (length to thickness), percent		5 max.
Fractured Faces, +4.75 mm		
One fractured face, percent		100 min.
Two fractured faces, percent		90 min.
Absorption, percent	AASHTO T 85	2 max.
Coarse and Fine Durability Index	AASHTO T 210	40 min.
Sulfate Soundness Loss, 5 cycles	AASHTO T 104	
Sodium sulfate, percent		15 max.
or Magnesium sulfate, percent		20 max.
Fine Aggregate:		
Crushed manufactured fines, percent		100 min.
Sulfate soundness loss, 5 cycles	AASHTO M 29	
Sodium sulfate, percent		15 max.
Liquid Limit	AASHTO T 89	25 max.

Table 5
Recommended Guidelines for SMA Cellulose Fiber Properties (after NAPA 1994)

Test	Requirement
Sieve Analysis	
Method A	
Alpine Sieve Analysis:	
Fiber Length	6 mm (0.25 in.) maximum
Passing 150 μm (No. 100) sieve	70 ± 10 percent
Method B	
Mesh Screen Analysis:	
Fiber Length	6 mm (0.25 in.) maximum
Passing 850 μm (No. 20) sieve	85 ± 10 percent
425 μm (No. 40) sieve	65 ± 10 percent
106 μm (No. 140) sieve	30 ± 10 percent
Ash Content	18 ± 5 percent non-volatiles
pH	7.5 ± 1
Oil Absorption	5 ± 1 times fiber weight
Moisture Content	< 5 percent by weight

Table 6
Recommended Guidelines for SMA Mineral Fiber Properties (after NAPA 1994)

Test	Requirement
Sieve Analysis	
Fiber Length	6 mm (0.25 in.) maximum
Thickness	5 μm (0.0002 in.) maximum mean test value
Shot Content	
250 μm (No. 60) sieve	95 percent minimum
63 μm (No. 230) sieve	65 percent minimum

Table 7
Recommended Guidelines for SMA Mixture Requirements (after NAPA 1994)

Design Parameters	Requirement
Marshal Compaction, blows per side	50
Marshall Stability, N	6,200 (1,400 lb) minimum
Marshall Flow, 0.25 mm	8-16
Voids Total Mix, percent	3-4
Voids Mineral Aggregate, percent	17 minimum
Draindown, percent at 1 hour	0.3 maximum
Asphalt Content, percent	6.0 minimum

Table 8
Typical SMA and Dense Graded Mixture Properties (after Mogawer and Stuart 1994)

Sieve Size	Gradation, Percent Passing		
	Dense Graded	SMA with Cellulose	SMA with Polymer
19 mm (3/4-in.)	100	100	100
12.5 mm (1/2-in.)	95	95	95
9.5 mm (3/8-in.)	82	71	71
4.75 mm (No. 4)	56	25	25
2.36 mm (No. 8)	39	20	20
1.18 mm (No. 16)	29	18	18
0.600 mm (No. 30)	21	16	16
0.300 mm (No. 50)	13.8	13	13
0.150 mm (No. 100)	9.1	12	12
0.075 mm (No. 200)	6.3	10	10
Asphalt Content, percent	4.5	6.7	6.8
VMA, percent	13.7	17.5	17.7
VTM, percent	4.0	3.0	3.0
Stability, N	17,066	7,116	8,286
Flow, 0.25 mm	10.7	14.3	18.2
Drainage Test			
German Method, percent loss		0.08	3.21
2.36 mm Sieve Method, percent loss		0.11	2.13
Diametral Modulus, MPa			
at -32°C (-25.6°F)	51,450	43,580	
at 0°C (32°F)	21,620	15,420	
at 25°C (77°F)	1,870	1,240	
at 40°C (104°F)	430	260	

Table 9
Properties of SMA and Dense-Graded Mixtures (after Brown 1992)

Properties	SMA					Dense-Graded
Asphalt content, percent	5.5	6.0	6.5	7.0	7.5	4.25
Unit weight, kg/m^3	2,444.7	2,438.2	2,443.1	2,441.4	2,436.6	2,455.9
VMA[1], percent	16	17.2	17.0	17.4	18.1	14.6
VTM[2], percent	2.9	3.0	1.6	0.8	0.03	4.5
Stability, N	6,897.5	7,195.7	7,520.5	8,072.3	7,867.6	14,524.8
Flow, 0.25 mm	16	14	16	21	24	14
Indirect Tensile, at 25°C, Kpa	983.2	939.8	953.5	1,003.9	982.5	1,703.0
Resilient Modulus, 10^3 Kpa						
at 4.4°C	15,499	17,795	15,175	16,030	15,775	20,864
at 25°C	2,392	2,151	2,034	1,606	1,896	7,198
at 40°C	807	752	1,269	683	717	3,640
Gyratory (GTM)						
GSI[3]	1.0	1.0	1.0	1.01	1.05	1.05
Shear, Kpa	263.4	271.0	313.0	277.9	250.3	259.2
Confined Creep						
Modulus, MPa	142.6	104.7	102.1	81.9	76.6	100.9
Permanent Strain, percent	0.58	0.79	0.81	1.01	1.06	0.82

[1] Voids in mineral aggregate (VMA).
[2] Voids in total mixture (VTM).
[3] Gyratory stability index (GSI).

Table 10
Test Plan for Asphalt Content Determination

Gradation	Mix Design		
	50 Blow	**75 Blow**	**Gyratory (GTM)**
Stone Mastic Asphalt			
Control			X
SMA	X	X	X

Gyratory compaction at 30 revolutions, 1,378.9 KPa (200 psi), 1 degree.
Compaction temperature = 146°C (295°F).

Table 11
Test Plan for Mixtures on Corps of Engineers Gyratory Testing Machine

Gradation	Asphalt Content, percent		
	J	K	L
SMA Maximum Compaction			
Control	X	X	X
SMA	X	X	X

Gyratory compaction of aggregate at 200 psi, 1 degree, 300 Rev.
J = optimum - 0.5 percent asphalt cement.
K = optimum percent asphalt cement.
L = optimum + 0.5 percent asphalt cement.
Compaction temperature = 295°F.

Table 12
Test Plan for Testing of Material Properties of Control and SMA Mixtures

Test	Control (G) J	Control (G) K	Control (G) L	SMA (G) J	SMA (G) K	SMA (G) L	SMA (H50) J	SMA (H50) K	SMA (H50) L
Creep									
at 25°C (77°F)		X			X				
at 40°C (104°F)	X	X	X	X	X	X		X	
Retained Stability		X			X			X	
Indirect Tensile									
at -18°C (0°F)		X			X				
at 25°C (77°F)	X	X	X	X	X	X		X	
at 40°C (104°F)		X			X				
ASTM D 2041	X	X	X	X	X	X	X	X	X

J = optimum - 0.5 percent asphalt cement.
K = optimum percent asphalt cement.
L = optimum + 0.5 percent asphalt cement.
G = Gyratory at 1,378.9 KPa (200 psi), 30 revolutions, 1 degree.
H50 = 50 blow hand hammer.

Table 13
Gradations of Test Mixtures

Sieve Size	Percent Passing			
	Control Mixture	Limits	SMA Desired	Actual
19 mm	100		100	100
12.7 mm	89	90-100	95	93.9
9.5 mm	82	54-80	67	68.2
4.75 mm	66	30-45	38	37.8
2.36 mm	53	20-30	25	24.5
1.18 mm	41	16-26	21	20.5
600 μm	31	13-25	19	17.9
300 μm	21	10-22	16	16.7
150 μm	13	9-19	14	15.2
75 μm	4.5	8-13	11	10.1

Table 14
Results of Marshall Tests[1] on Control and SMA Mixtures

Type of Mixture	Type of Compaction	Asphalt Cement Content (%)	Specific Gravity			Voids[2] (%)			Unit Weight (Kg/m³)	Stability (N)	Flow (0.25 mm)
			Actual	Theoretical							
				Calc.[3]	D 2041[4]	Total Mix	Agg.[5] Only	Filled			
SMA	50 H[6]	4.5	2.494	2.618	2.579	4.7	15.5	69.7	2,494.3	8,148.0	10
		5.0	2.503	2.510	2.563	3.7	15.7	76.4	2,502.3	8,401.6	10
		5.5	2.490	2.577	2.536	3.4	16.6	79.5	2,489.5	8,343.8	13
		6.0	2.506	2.557	2.553	2.0	16.5	87.9	2,505.5	7,711.9	14
		6.5	2.501	2.538	2.508	1.5	17.1	91.2	2,500.1	7,266.9	17
SMA	75 H[7]	4.5	2.506	2.618	2.579	4.3	15.2	71.7	2,505.5	8,868.9	9
		5.0	2.519	2.598	2.563	3.0	15.1	80.1	2,518.3	10,395.2	11
		5.5	2.513	2.577	2.536	2.5	15.8	84.2	2,511.9	9,055.8	11
		6.0	2.522	2.557	2.553	1.4	16.0	91.3	2,521.5	8,793.2	14
		6.5	2.498	2.538	2.508	1.6	17.2	90.7	2,497.5	7,849.8	16
SMA	GTM-75[8]	4.5	2.496	2.618	2.579	4.7	15.5	69.7	2,495.9	7,262.4	11
		5.0	2.492	2.598	2.563	4.1	16.1	74.5	2,491.1	6,581.6	12
		5.5	2.505	2.577	2.536	2.8	16.1	82.6	2,503.9	7,409.3	13
		6.0	2.494	2.557	2.553	2.5	16.9	85.2	2,492.7	7,146.7	12
		6.5	2.501	2.538	2.508	1.5	17.1	91.2	2,500.7	7,565.0	12
Control[9]	GTM-75[8]	4.0	2.474	2.643	--	6.4	15.9	59.7	2,476.7	7,400.4	8
		4.5	2.483	2.622	--	5.3	16.1	67.1	2,482.3	8,655.3	10
		5.0	2.495	2.601	--	4.1	16.1	74.5	2,507.1	9,669.9	10
		5.5	2.512	2.581	--	2.7	16.0	83.1	2,508.7	10,555.4	11
		6.0	2.525	2.561	--	1.4	16.0	91.2	2,511.9	10,791.3	12

1 Each value given is the average of three specimens.
2 Void parameters determined using calculated theoretical specific gravity values.
3 Based on apparent specific gravity.
4 Obtained according to ASTM D 2041.
5 Specific gravity of AC 20 = 1.039.
6 50-blow Marshall compaction effort.
7 75-blow Marshall compaction effort.
8 GTM compaction equivalent to 75-blow Marshall (30 revolutions, 1,379.0 kPa at 1°)
9 Control mix is a conventional 3/4-in. maximum aggregate size, dense graded high-pressure (75-blow) mixture.

Table 15
Results of Retained Stability Tests[1] on SMA

Type of Mixture	Type of Compaction	Asphalt Cement Content (%)	Specific Gravity		Voids (%)			Unit Weight (kg/m^3)	Stability (N)	Flow (0.25 mm)
			Actual[2]	Theoretical[3]	Total Mix	Aggregate Only	Filled			
SMA	GTM-75	6.0	2.483	2.557	2.9	17.2	83.1	2,483.1	7,186.8	16
SMA	GTM-75	6.0	2.484	2.557	2.9	17.2	83.1	2,483.1	7,289.1[4]	14

1 Each value given is the average of three specimens.
2 Calculated using apparent specific gravity.
3 Obtained according to ASTM D 2041.
4 All 3 specimens soaked at 140°F for 24 hr.

Table 16
Physical Properties of Tensile Test Specimens

Test[1] No.	Type of Mixture	Type of Compaction	Asphalt Content (percent)	Specimen Thickness (cm)	Specific Gravity (-)	Biitumen Content by Volume (percent)	Voids[2] (percent)			Unit Weight (kg/m^3
							Total Mix	Agg Only	Filled	
SH1	SMA	50 H	6.0	6.492	2.477	14.3	3.1	17.4	82.2	2,476.7
SH2	SMA	50 H	6.0	6.441	2.483	14.3	2.9	17.2	83.1	2,483.1
SH3	SMA	50 H	6.0	6.414	2.470	14.3	3.4	17.7	80.8	2,468.7
SJ1	SMA	GTM-75	5.5	6.574	2.478	13.1	3.8	16.9	77.5	2,476.7
SJ2	SMA	GTM-75	5.5	6.530	2.489	13.2	3.4	16.6	79.5	2,487.9
SJ3	SMA	GTM-75	5.5	6.546	2.466	13.1	4.3	17.4	75.3	2,465.5
CJ1	Control	GTM-75	4.6	6.368	2.467	10.9	5.2	16.1	67.7	2,465.5
CJ2	Control	GTM-75	4.6	6.327	2.466	10.9	5.2	16.1	67.7	2,465.5
CJ3	Control	GTM-75	4.6	6.299	2.472	10.9	5.0	15.9	68.6	2,471.9
SK1	SMA	GTM-75	6.0	6.586	2.468	14.3	3.5	17.8	80.3	2,467.1
SK2	SMA	GTM-75	6.0	6.523	2.473	14.3	3.3	17.6	81.3	2,471.9
SK3	SMA	GTM-75	6.0	6.505	2.477	14.3	3.1	17.4	82.2	2,476.7
SK4	SMA	GTM-75	6.0	6.558	2.467	14.2	3.5	17.7	80.2	2,465.5
SK5	SMA	GTM-75	6.0	6.454	2.497	14.4	2.3	16.7	86.2	2,495.9
SK6	SMA	GTM-75	6.0	6.487	2.492	14.4	2.5	16.9	85.2	2,491.1
SK7	SMA	GTM-75	6.0	6.523	2.476	14.3	3.2	17.5	81.7	2,475.1
SK8	SMA	GTM-75	6.0	6.487	2.490	14.4	2.6	17.0	84.7	2,489.5
SK9	SMA	GTM-75	6.0	6.510	2.490	14.4	2.6	17.0	84.7	2,489.5

(Continued)

1 'S' and 'C' stand for SMA and control mixes, respectively; 'K' designates samples at optimum asphalt content and 'J' and 'L' are 0.5 percent below and above, respectively.
2 Void parameters determined using calculated theoretical specific gravity values.

Table 16 (Concluded)

Test[1] No.	Type of Mixture	Type of Compaction	Asphalt Content (percent)	Specimen Thickness (cm)	Specific Gravity (-)	Bitumen Content by Volume (percent)	Voids[2] (percent)			Unit Weight (kg/m^3)
							Total Mix	Agg Only	Filled	
CK1	Control	GTM-75	5.1	6.434	2.461	12.1	4.7	16.8	72.0	2,460.7
CK2	Control	GTM-75	5.1	6.347	2.465	12.1	4.5	16.6	72.9	2,463.9
CK3	Control	GTM-75	5.1	6.317	2.473	12.1	4.2	16.3	74.2	2,471.9
CK4	Control	GTM-75	5.1	6.454	2.467	12.1	4.5	16.6	72.9	2,467.1
CK5	Control	GTM-75	5.1	6.358	2.483	12.2	3.8	16.0	76.3	2,481.5
CK6	Control	GTM-75	5.1	6.210	2.490	12.2	3.6	15.8	77.2	2,489.5
CK7	Control	GTM-75	5.1	6.317	2.482	12.2	3.9	16.1	75.8	2,479.9
CK8	Control	GTM-75	5.1	6.388	2.472	12.1	4.3	16.4	73.8	2,470.3
CK9	Control	GTM-75	5.1	6.383	2.484	12.2	3.8	16.0	76.3	2,483.1
SL1	SMA	GTM-75	6.5	6.490	2.488	15.6	2.0	17.6	88.6	2,486.3
SL2	SMA	GTM-75	6.5	6.449	2.485	15.5	2.1	17.6	88.1	2,484.7
SL3	SMA	GTM-75	6.5	6.449	2.485	15.5	2.1	17.6	88.1	2,484.7
SL4	SMA	GTM-75	6.5	6.391	2.500	15.6	1.5	17.1	91.2	2,499.1
CL1	Control	GTM-75	5.6	6.320	2.496	13.5	2.6	16.1	83.9	2,495.9
CL2	Control	GTM-75	5.6	6.307	2.496	13.5	2.6	16.1	83.9	2,495.9
CL3	Control	GTM-75	5.6	6.396	2.487	13.4	2.9	16.3	82.2	2,486.3

Table 17
Summary of Indirect Tensile Test Results

Test No.	Temperature °C	Tensile Strength (kPa)	Standard Deviation of Tensile Strength Values (kPa)	Elastic Modulus (MPa/m)	Standard Deviation of Elastic Modulus Values (MPa/m)	Deflection at Maximum Load (cm)	Standard Deviation of Maximum Load Values (cm)
SH1	25	638.5		590.2		0.157	
SH2	25	599.2		600.5		0.145	
SH3	25	568.8		530.5		0.165	
Average		601.9	35.2	573.7	37.8	0.156	0.0102
SJ1	25	601.2		590.0		0.147	
SJ2	25	660.5		653.2		0.160	
SJ3	25	641.2		516.2		0.173	
Average		634.3	30.3	586.5	68.6	0.160	0.0127
CJ1	25	799.8		599.5		0.152	
CJ2	25	721.2		552.2		0.175	
CJ3	25	734.3		558.9		0.145	
Average		751.5	42.1	570.2	25.5	0.157	0.0157
SK1	25	493.0		451.2		0.152	
SK2	25	565.4		568.1		0.152	
SK3	25	515.7		492.0		0.157	
Average		524.7	37.2	503.8	59.3	0.154	0.0030
SK4	40	167.5		173.4		0.175	
SK5	40	177.2		149.1		0.165	
SK6	40	188.9		197.6		0.140	
Average		177.9	11.0	173.4	24.2	0.160	0.0183
SK7	-17.8	3,692.1		5,111.6		0.079	
SK8	-17.8	4,354.7		5,007.7		0.097	
SK9	-17.8	4,419.5		5,399.6		0.097	
Average		4,126.5	452.3	5,105.3	259.7	0.090	0.0102
CK1	40	148.2		160.3		0.137	
CK2	40	148.9		168.1		0.109	
CK3	40	171.0		175.9		0.123	
Average		155.8	13.1	168.1	7.8	0.123	0.0140

(Continued)

Table 17 (Concluded)							
Test No.	**Temperature °C**	**Tensile Strength (kPa)**	**Standard Deviation of Tensile Strength Values (kPa)**	**Elastic Modulus (MPa/m)**	**Standard Deviation of Elastic Modulus Values (MPa/m)**	**Deflection at Maximum Load (cm)**	**Standard Deviation of Maximum Load Values (cm)**
CK4	-17.8	3,448.8		4,369.1		0.089	
CK5	-17.8	3,706.6		5,091.6		0.081	
CK6	-17.8	3,421.2		4,697.1		0.089	
Average		3,525.3	157.2	4,719.2	361.8	0.086	0.0043
CK7	25	779.1		604.7		0.152	
CK8	25	755.7		562.3		0.183	
CK9	25	813.6		590.8		0.185	
Average		782.6	29.0	585.9	21.6	0.173	0.0183
SL1	25	612.3		578.2		0.157	
SL2	25	616.4		598.2		0.157	
SL3	25	597.8		638.4		0.142	
SL4	25	608.1		574.5		0.155	
Average		608.8	8.3	601.4	26.8	0.153	0.0074
CL1	25	917.7		808.6		0.152	
CL2	25	774.3		765.0		0.150	
CL3	25	758.4		644.8		0.157	
Average		817.0	87.6	739.4	84.8	0.153	0.0038

Table 18
Physical Properties of Creep-Rebound Test Specimens

Test[1] No.	Type of Mixture	Type of Compaction	Asphalt Content (percent)	Total Specimen Height (cm)	Specimen Specific Gravity	Asphalt Content by Volume (percent)	Voids (percent)			Unit Weight (kg/m^3)
							Total Mix	Aggregate Only	Filled	
CJ1R	Control	GTM-75	4.6	19.14	2.460	10.9	5.5	16.3	66.6	2,459.4
CJ2R	Control	GTM-75	4.6	18.81	2.475	11.0	4.9	15.8	69.2	2,474.1
CK1R	Control	GTM-75	5.1	18.98	2.483	12.2	3.8	16.0	76.1	2,482.3
CK2R	Control	GTM-75	5.1	18.93	2.486	12.2	3.7	15.9	76.6	2,485.0
CK3R	Control	GTM-75	5.1	19.00	2.484	12.2	3.8	16.0	76.3	2,483.4
CK4R	Control	GTM-75	5.1	18.96	2.476	12.2	4.1	16.3	74.7	2,474.9
CK5R	Control	GTM-75	5.1	19.01	2.471	12.1	4.3	16.4	73.8	2,470.1
CK6R	Control	GTM-75	5.1	18.85	2.479	12.2	3.8	16.1	75.4	2,478.5
CL1R	Control	GTM-75	5.6	19.07	2.493	13.4	2.7	16.1	83.3	2,492.1
CL2R	Control	GTM-75	5.6	18.88	2.501	13.5	2.4	15.9	84.9	2,499.9
CL3R	Control	GTM-75	5.6	18.80	2.496	13.5	2.6	16.0	84.0	2,495.3
SH1R	SMA	50 H	6.0	19.53	2.485	14.4	2.8	17.2	83.6	2,484.1
SH2R	SMA	50 H	6.0	19.66	2.472	14.3	3.3	17.6	81.2	2,471.1
SH3R	SMA	50 H	6.0	19.67	2.482	14.3	2.9	17.3	83.1	2,481.2
SJ1R	SMA	GTM-75	5.5	19.78	2.488	13.2	3.5	16.6	79.1	2,486.6
SJ2R	SMA	GTM-75	5.5	19.70	2.480	13.1	3.8	16.9	77.8	2,479.4
SJ3R	SMA	GTM-75	5.5	19.60	2.485	13.2	3.6	16.7	78.6	2,483.9
SK1R	SMA	GTM-75	6.0	19.37	2.485	14.4	2.8	17.2	83.6	2,483.9
SK2R	SMA	GTM-75	6.0	19.50	2.479	14.3	3.0	17.4	82.5	2,478.3
SK3R	SMA	GTM-75	6.0	19.54	2.488	14.4	2.7	17.1	84.3	2,487.6
SL1R	SMA	GTM-75	6.5	19.47	2.489	15.6	1.9	17.5	89.0	2,488.4
SL2R	SMA	GTM-75	6.5	19.38	2.488	15.6	2.0	17.5	88.8	2,487.3
SL3R	SMA	GTM-75	6.5	19.48	2.485	15.6	2.1	17.6	88.2	2,484.4
SK1R	SMA	GTM-75	6.0	19.52	2.481	14.3	3.0	17.3	82.7	2,479.7
SK2R	SMA	GTM-75	6.0	19.43	2.489	14.4	2.7	17.0	84.3	2,487.9

[1] 'S' and 'C' designates SMA and control mixtures respectively; 'K' designates samples at optimum asphalt content and 'J' and 'L' are 0.5 percent below and above, respectively; and 'R' designates mixes used for creep-rebound testing.

Table 19
Equation Coefficients and Coefficients of Correlation for Creep-Rebound Tests

Test[1] No.	Test Temperature (°C)	Asphalt Content (percent)	Axial Pressure (kPa)	Creep			Rebound		
				1000xa	b	R^2	1000xa	b	R^2
CJ1R	40	4.6	103.4	1.15802	0.17015	0.91354	4.10816	0.05101	0.98255
CJ2R	40	4.6	103.4	2.47829	0.10485	0.94706	5.8643	0.03941	0.93252
CK1R	40	5.1	103.4	1.94407	0.11639	0.94228	4.91578	0.05459	0.96062
CK2R	40	5.1	103.4	3.29269	0.12495	0.77344	6.95912	0.04651	0.97078
CK3R	40	5.1	103.4	1.53753	0.16463	0.93895	5.47936	0.04826	0.96021
CK4R	25	5.1	275.8	2.68865	0.18560	0.96678	11.2286	0.04427	0.87665
CK5R	25	5.1	275.8	3.29269	0.12495	0.93511	8.23385	0.03394	0.94878
CK6R	25	5.1	275.8	1.52302	0.17681	0.99772	6.54766	0.03172	0.92094
CL1R	40	5.6	103.4	1.42556	0.11647	0.92510	3.84910	0.10284	0.88707
CL2R	40	5.6	103.4	1.13535	0.17395	0.90360	4.17274	0.04735	0.98804
CL3R	40	5.6	103.4	1.08390	0.21467	0.95682	5.54274	0.05597	0.97887
SH1R	40	6.0	103.4	1.03043	0.21705	0.96007	7.36985	0.01270	0.92273
SH2R	40	6.0	103.4	0.90439	.030976	0.94342	9.92574	0.03189	0.99221
SH3R	40	6.0	103.4	0.66273	0.32908	0.95720	7.67553	0.02124	0.97935
SJ1R	40	5.5	103.4	1.63005	0.18801	0.95477	6.87499	0.03716	0.96535
SJ2R	40	5.5	103.4	1.23663	0.22376	0.95272	7.31551	0.04505	0.97174
SJ3R	40	5.5	103.4	1.27680	0.14717	0.95028	4.55523	0.10790	0.92499
SK1R	40	6.0	103.4	1.56968	0.11310	0.97210	3.97639	0.06137	0.96393
SK2R	40	6.0	103.4	1.41792	0.13617	0.94344	4.35672	0.00813	0.65481
SK3R	40	6.0	103.4	0.48320	0.21654	0.71123	2.10999	0.01121	0.79285
SL1R	40	6.5	103.4	0.99370	0.22354	0.84153	4.60934	0.031194	0.97897
SL2R	40	6.5	103.4	1.41193	0.20300	0.90116	6.08081	0.023935	0.96942
SL3R	40	6.5	103.4	1.07027	0.22265	0.90623	5.18464	0.01710	0.96109
SK1R	25	6.0	275.8	1.78351	0.30950	0.98334	19.75260	0.02897	0.91962
SK2R	25	6.0	275.8	0.77011	0.31059	0.99337	9.16659	0.03829	0.92512

[1] 'S' and 'C' designates SMA and control mixtures respectively; 'K' designates samples at optimum asphalt content and 'J' and 'L' are 0.5 percent below and above, respectively; and 'R' designates mixes used for creep-rebound testing.

Table 20
JMF Developed at WES from Samples Submitted by Contractor

Sieve Size	Specification Limits	Job Mix Formula	19 mm 7%	12.7 mm 37%	9.5 mm 35%	Sand 13%	Fly Ash 8%
25.4 mm							
19 mm	100	100	100	100	100	100	100
12.7 mm	80-95	87.5	7.9	85.2	100	100	100
9.5 mm	60-75	67.5	1.5	28.7	98.9	100	100
4.75 mm	25-34	29.5	1.0	1.2	24.0	93.7	100
2.36 mm	18-24	21.0	0.9	1.0	6.4	73.4	100
1.18 mm	14-20	17.0	0.8	0.9	4.3	60.0	100
600 μm	13-18	15.5	0.7	0.8	3.6	44.9	100
300 μm	11-16	13.5	0.6	0.7	3.2	28.7	98.5
150 μm	10-14	12.0	0.5	0.6	2.7	16.6	95.7
75 μm	8-12	10.0	0.3	0.5	2.2	11.0	89.0
Percent Asphalt	---	7.3[1]					
Grade Asphalt	---	AR-4000					
Stability (Marshall) N	7,120 minimum	6,239.0[2]					
Flow 0.25 mm	8-16	12.8					
Percent Voids Total Mix	3-5	3.5					
Percent Voids Filled	---	82.8					
Density kg/m^3	2,342.1						
Theo Density kg/m^3	2,425.4						
Retained Stability (percent)	94.6						
Mixing/Compaction Temperature (°C)	149/140						

[1] Use 7.2 percent AC for JMF.
[2] Stability is below the 7,120 N minimum given in specification; however, it is recommended that this JMF be used.

Table 21
Results of Aggregate Tests for SMA at Edwards AFB, CA

Aggregate Property	Specification Requirement	19 mm Stone	12.7 mm Stone	9.5 mm Stone	Sand	Fly Ash
Specific Gravity ± 4.75 mm	none	2.71/---	2.73/---	2.73/2.71	--- /2.73	--- /2.71
Absorption (percent water) ± 4.75 mm	none	0.8/---	1.0/---	1.1/1.0	--- /1.7	---
Fractured Face (percent) ± 4.75 mm	minimum of 90 percent	71.4/---	91.5/---	95/100	---/100	---
Flat Particles[1] (% of 3:1)	minimum of 20 percent	12.7 mm 9.5 mm 4.75 mm	12.7 mm 9.5 mm 4.75 mm	4.75 mm - 20.0	---	---
Elongated Particles[1] (% of 3:1)	maximum of 20 percent	none	12.7 mm 9.5 mm 4.75 mm	4.75 mm - 1.9	---	---
Magnesium Sulfate Soundness (%) (4.75 mm sieve) (600 μm sieve)	maximum of 18 percent	7.5	4.2	5.4	11.9	---

[1] Contained no 5:1 particles.

Table 22
Manufacturer's Data on Cellulose Fibers

Property	Requirement
Fiber Type[1]	Cellulose
Fiber length	0.25 in. (maximum)
Sieve Analysis	
a. Alpine Sieve Method passing 150 μm sieve	60-80 percent
b. Ro-Tap Sieve Method passing 850 μm sieve passing 425 μm sieve passing 75 μm sieve	 80 - 90 percent 45 - 85 percent 5 - 40 percent
Ash content	18 (+/-) percent non-volatile
pH	7.5 (+/- 1.0)
Oil absorption (times fiber weight)	5.0 (+/- 1.0)
Moisture content	5.0 percent (maximum)
Fiber Type[2]	Cellulose pellets
Pellet size	0.256 cm^3 (maximum)
Asphalt	25-80 pen.

[1] Cellulose Fibers: Cellulose fibers shall be added at a dosage rate between 0.2 and 0.4 percent by weight of the total mix as approved by the engineer.
[2] Cellulose Pellets: Cellulose pellets shall consist of a 50/50 blend of cellulose fiber and asphalt cement and shall be added at a dosage rate between 0.4 and 0.8 percent by weight of the total mix. The cellulose used shall comply with requirements as given in this table.

Table 23
Gradation of Aggregates from Stockpile Samples

Stockpile	19 mm	12.7 mm	9.5 mm	Sand
Sieve Size (% passing)				
25.4 mm	---	---	---	---
19 mm	100	100	---	---
12.7 mm	2.4	88.1	100	---
9.5 mm	---	26.2	97.6	100
4.75 mm	---	0.3	16.4	97
2.36 mm	---	---	3.9	79.8
1.18 mm	---	---	---	58.5
600 μm	---	---	---	35.9
300 μm	---	---	---	18.5
150 μm	---	---	---	10.6
75 μm	---	---	---	6.8

Table 24
Gradation of Hot-Bin Samples after Cold Feed Calibration[1]; Including Percentages of Each Required and New Blended Gradation

Bin Number	1[2]	2	3	4	Fly Ash	Combined Final Gradation
Date Sampled	8/3/93	8/3/93	8/3/93	8/3/93		
% of Bin for JMF	14.5	20.5	42.0	14.0	9.0	
Sieve size (% passing)						
19 mm	---	---	100	100	---	100
12.7 mm	---	100	96.0	21	---	87
9.5 mm	100	99.7	53.7	0.2	---	67
4.75 mm	99.2	39.9	2.1	0.1	---	32
2.36 mm	78.1	0.5	0.3	---	---	21
1.18 mm	54.0	0.1	0.2	---	---	17
600 μm	31.1	---	---	---	100	14
300 μm	11.6	---	---	---	99.0	11
150 μm	4.2	---	---	---	97.0	9
75 μm	2.2	---	---	---	89.0	8.3

[1] Stockpiles were tested for gradation at the plant and the percentages for the listed materials changed from 7, 37, 35, 13, and 8 to 6, 35, 35, 15, and 9.
[2] Washed gradation.

Table 25
Results of Testing of SMA Mixture Placed at Edwards AFB, CA

Sample No.	Test[1] Mixture	1	2	3	4	5	6	7
Date Sampled (Time)	8/3/93	8/5/93 (0800)	8/5/93 (1015)	8/5/93 (1400)	8/5/93 (1545)	8/6/93 (0820)	8/6/93 (1015)	8/6/93 (1315)
Placement Temp (°C)	---	163	177	149	---	163	135[2]	135[2]
Sieve Size (% Passing)								
25.4 mm	100	100	---	---	---	---	---	---
19 mm	99.2	98.5	100	100	100	100	100	100
12.7 mm	85.5	89.7	90.6	89.6	86.8	94.2	88.5	83.7
9.5 mm	68.4	72.1	71.8	68.6	64.6	64.9	66.3	64.7
4.75 mm	32.0	33.2	34.4	34.2	30.7	29.2	33.1	31.6
2.36 mm	20.9	21.1	21.6	22.8	20.7	19.8	22.3	22.3
1.18 mm	18.0	17.3	17.8	19.2	17.6	16.7	18.8	19.2
600 μm	16.3	14.5	15.3	17.1	15.2	14.3	16.2	16.5
300 μm	14.4	11.7	13.2	15.1	12.9	12.0	13.7	13.9
150 μm	11.7	9.5	11.5	13.1	10.9	10.1	11.5	11.9
75 μm	8.7	7.5	9.8	10.9	9.0	8.4	9.4	9.9
% Asphalt	5.8	6.5	6.5	6.5	5.9	6.0	7.0	5.9
Stability (N)	8,713	7,605	8,063	8,126	7,289	7,449	6,595	8,802
Flow (0.25 mm)	21	17	18	17	13	14	15	11
% Voids Filled	73.7	82.0	82.5	82.0	75.1	77.2	84.8	77.8
% Voids Total Mix (Calc/D 2041)	4.9/1.6	3.3/.08	3.2/0.4	4.2/0.0	4.3/0.0	2.7/---	4.5/---	---/---
Max Theor Density (Calc/D 2041)	2.483/2.401	2.438/2.376	2.438/2.335	2.460/2.330	2.456/2.311	2.420/---	2.460/---	---/---
Specific Gravity	2.362	2.357	2.360	2.357	2.350	2.355	2.349	2.364
Density (Kg/m^3)	2,361.3	2,356.5	2,359.7	2,356.5	2,348.5	2,354.9	2,348.5	2,362.9

1 Results used to develop a new JMF, selected 7.1 percent asphalt by weight of total mix.
2 Required reheating in oven prior to compaction.

Table 26
Test Results on 1994 SMA Field Cores

Size of Sieve	Core No. 1	Core No. 2	Core No. 3	Core No. 4	Core No. 5	Core No. 6
1 in.	100	100	100	---	100	---
1/2 in.	87.7	90.0	92.5	---	90.6	---
3/8 in.	67.7	63.6	70.4	---	71.1	---
No. 4	37.6	33.0	38.5	---	38.8	---
No. 8	25.7	24.2	26.6	---	26.7	---
No. 16	21.2	20.2	21.9	---	22.3	---
No. 30	18.1	17.5	18.9	---	19.2	---
No. 50	15.3	15.0	16.2	---	16.5	---
No. 100	12.8	12.6	13.8	---	13.9	---
No. 200	10.7	10.6	11.4	---	11.5	---
Percent Asphalt (extraction)	6.7	6.6	6.8	---	6.9	---
Pen Asphalt (77°F/39°F)	35/11	21/5	35/5	---/---	35/7	---/---
Density-lbs/cu ft	146.0	139.4	147.2	145.9	147.0	146.9
Theo density-lbs/cu ft	---	---	---	147.6	---	148.1
Viscosity P/CS 140°F/275°F	3460/344	6714/451	3365/316	---/---	2174/336	---/---
Asphalt specific gravity	1.027	1.028	1.029	---/---	1.025	---/---

Table 27
Comparison of SMA Data, As Constructed Versus One Year Later

Sieve Size (% Passing)	Spec. Limits	Job Mix Formula	Average[1] As Constructed Data	Average[2] 1994 Core Samples	Currently Recommended Gradation[3]
25.4 mm	100	100	100	100	---
19 mm	100	99.2	99.6	100	100
12.7 mm	80-95	85.5	89.2	90.2	85-95
9.5 mm	60-75	68.4	69.3	68.2	75 maximum
4.75 mm	25-35	32.0	33.1	37.0	20-28
2.36 mm	18-24	20.9	21.6	25.8	16-24
1.18 mm	14-20	18.0	18.0	21.4	---
600 μm	13-18	16.3	15.5	18.4	12-16
300 μm	11-16	14.4	13.2	15.8	12-15
150 μm	10-14	11.7	11.3	13.3	---
75 μm	8-12	8.7	9.3	11.1	8-10
% Asphalt	---	5.8	6.4	6.75	---
Stability (N)	7,120 (min)	8,713	7,770	---	---
Flow (0.25 mm)	8-16	21	16	---	---
% Voids filled	---	73.7	80.4	---	---
% Voids (Calc/ D 2041)	3-5	4.9/1.6	3.6/0.0[4]	---/1.7[5] ---/0.9[6]	---
Max theo density (Calc/ D 2041)	---	2.483/ 2.401	2.444/ 2.353	---/ 2.370[7]	---
Specific gravity	---	2.362	2.356	2.330[5] 2.349[6]	---
Density (Kg/m^3)	---	2,361.3	2,354.9 2,329.3[8]	2,329.3[5] 2,348.5[6]	---
Asphalt S.G.	---	1.0185	---	1.027	---
Viscosity 60°C/135°C (Pa·s /cm^2/s	---	Ori--/40.6 RTF--/3.8	---	392.8/--- 36.2/---	---

1 Average results of 4 laboratory samples taken 8/15/93.
2 Average of 4 field cores for gradation.
3 Recommended gradation guideline by NAPA, 1994.
4 The voids calculated using D 2041 would have been negative for 2 of the samples.
5 Average of 6 field cores for density.
6 Average of 5 field cores (without No. 2) for density.
7 Average of field cores 4 and 6.
8 Average of 6 field cores taken from SMA placed 8/15/93.

Table 28
Mixture Properties of SMA Used in Demonstration at RAF Lakenheath

Sieve Designation			Specification Limits percent passing	Job Mix Formula percent passing	Field Test Results	
Standard	in.	Alternative			Test Section[1] percent passing	Demonstration Site[2] percent passing
20[3] mm	(0.7894)	--	100	--	--	--
14[3] mm	(0.551)	--	--	100	100	100
12[3] mm	(0.4697)	--	90-100	--	--	--
10[3] mm	(0.391)	--	--	95	94	94
8 mm	(0.3125)	5/16	48-75	--	--	--
6.3 mm	(0.250)	1/4	--	39	36	36.5
5[3] mm	(0.1968)	--	30-50	33	28.5	30.5
4 mm	(0.1570)	No. 5	28-40	--		
2.36 mm	(0.0937)	No. 8	--	26	23	26
2.0 mm	(0.0787)	No. 10	20-30	--		
1.18 mm	(0.0469)	No. 16	--	20	18	20.5
1.0 mm	(0.0394)	No. 18	15-28	--		
600 μm	(0.0234)	No. 30	--	17	14	17
500 μm	(0.0197)	No. 35	12-23	--		
300 μm	(0.0117)	No. 50	--	14	11	14
150 μm	(0.0059)	No. 100	--	12	8.8	11
125 μm	(0.0049)	No. 125	9-15	--		
75 μm	(0.0029)	No. 200	7-12	9	7.2	9.2
Asphalt percent			6.5 min.	6.8	6.4[4]	6.75[5]
Grade Asphalt			50 pen	50 pen		
Marshall Stability[6] kN (Lbf)			6.86 (1,540)	7.4 (1,664)		
Percent Voids Total Mix			3-4	2.4		
Density Kg/m^3 (lb/ft^3)						
Theoretical Density Kg/m^3 (lb/ft^3)				2,460		
Binder Drainage percent				0.1		
Mixing Temperature °C (°F)				150 (328)		
Compaction Temperature °C (°F)				70-90 (180-220)		
Cellulose Fibers percent			2-4	3		

1 Average of 3 tests.
2 Average of 2 tests.
3 British standard sieve size, remaining sieves are standard according to ASTM E-11.
4 Target asphalt content was 6.6 percent.
5 Target asphalt content was 6.8 percent.
6 60-blow Marshall compaction.

Table 29
Properties of the Aggregate Used in the SMA Placed at Lakenheath

Property Type	Materials[1] Coarse Stone Granite Specification	Materials[1] Coarse Stone Granite Actual	Materials[1] Sand Crushed Flint Specification	Materials[1] Sand Crushed Flint Actual
Specific Gravity	--	2.81	--	2.65
Flakiness Index, percent	25 Max	21	--	--
Aggregate Crushing Value, percent	16 Max	15	--	17
Absorption, percent	1.5 Max	0.7	2 Max	1.1
Aggregate Soundness, percent	18 Max	9.7	--	--

[1] Ground limestone and cellulose fibers used as stabilizing additives.

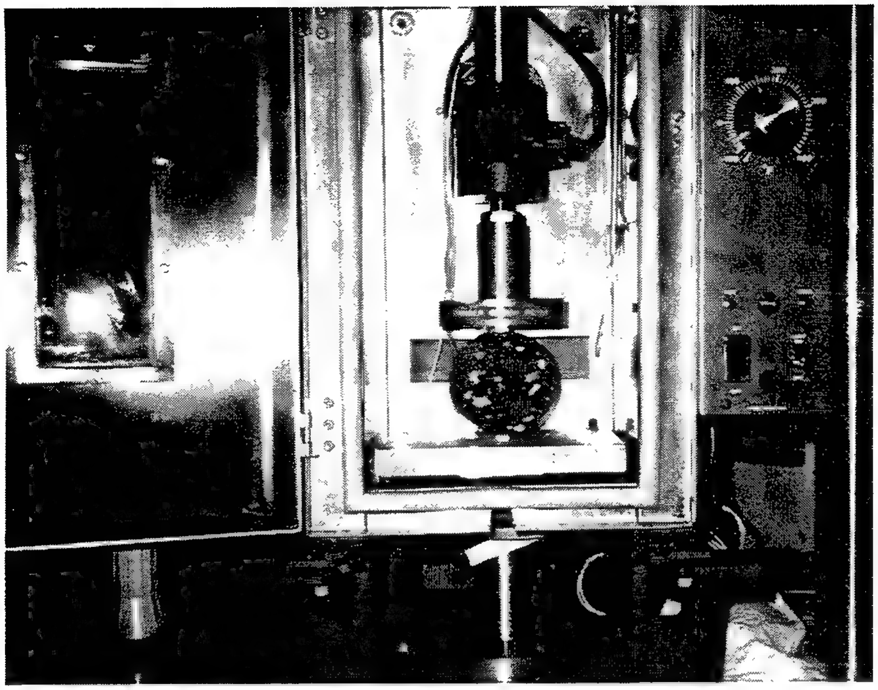


Photo 1. Indirect tensile test apparatus loaded with a specimen for testing

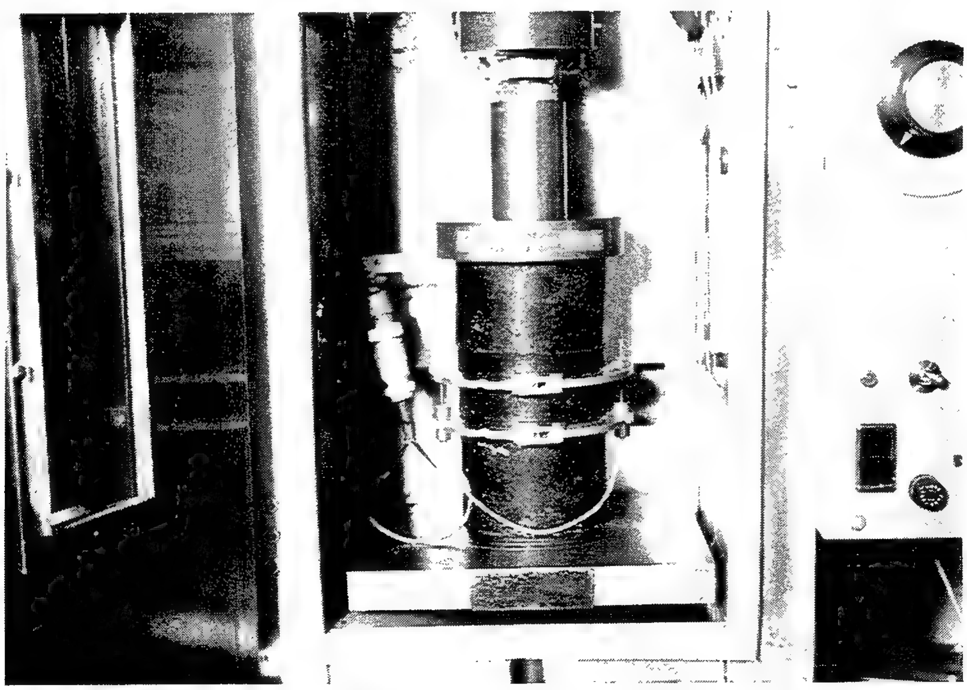


Photo 2. Creep-bound test apparatus containing 3 stacked cylindrical specimens for testing

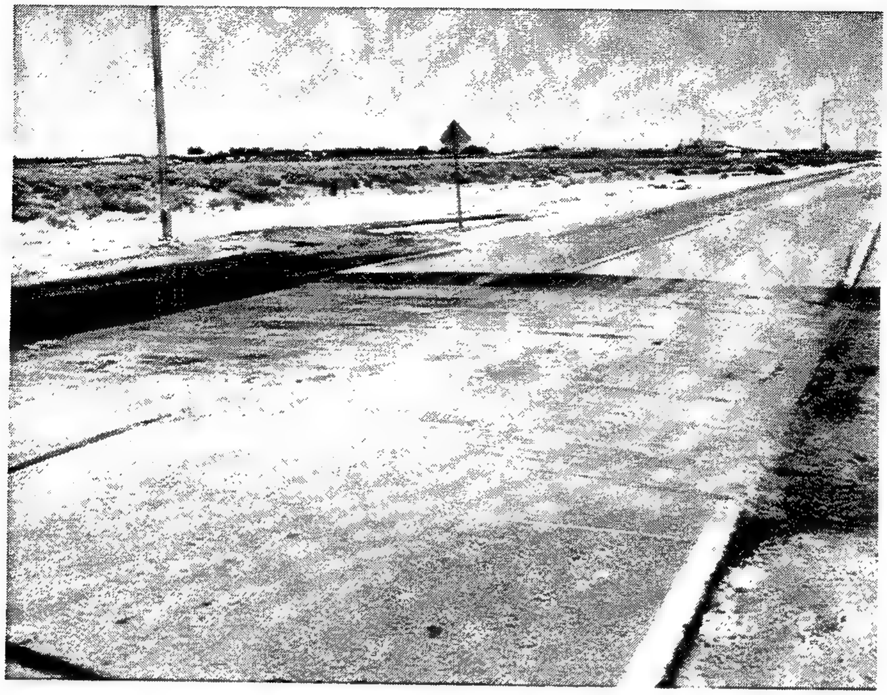


Photo 3. Cold milled section of Lancaster Blvd. removed for transition between SMA and existing pavement surface (looking south)

Photo 4. Base course added as part of intersection reconfiguration (looking north)

Photo 5. View of typical pavement condition prior to SMA overlay, note utility cut through existing pavement

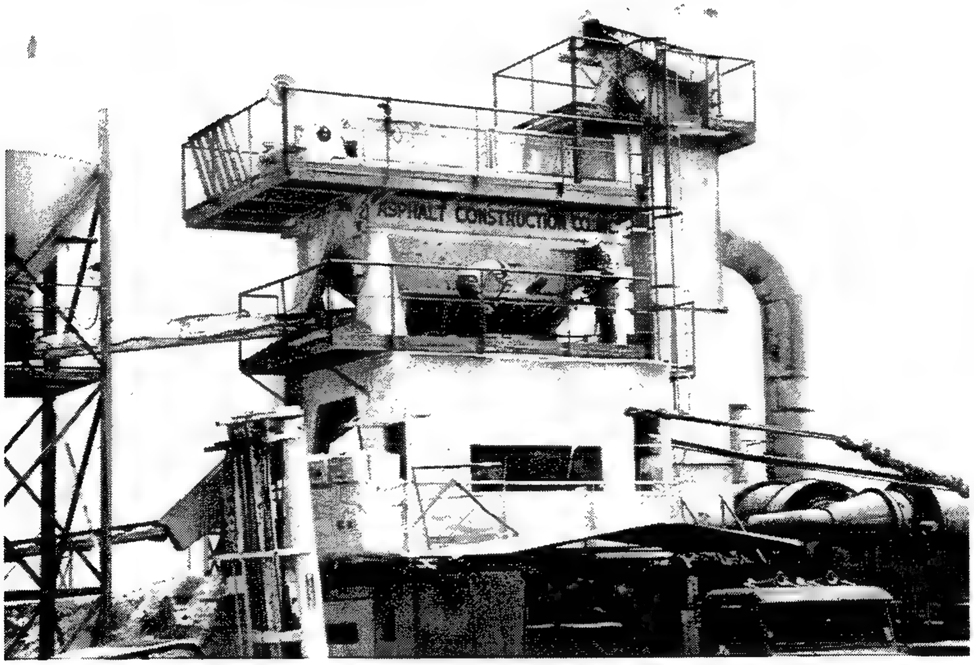


Photo 6. A pallet of cellulose fibers placed for adding directly into the pugmill, note-hot-bin samples being obtained from above

Photo 7. View of aggregates in the four cold feeds

Photo 8. Placing intermediate course of SMA over base course on Lancaster Blvd. (looking north)

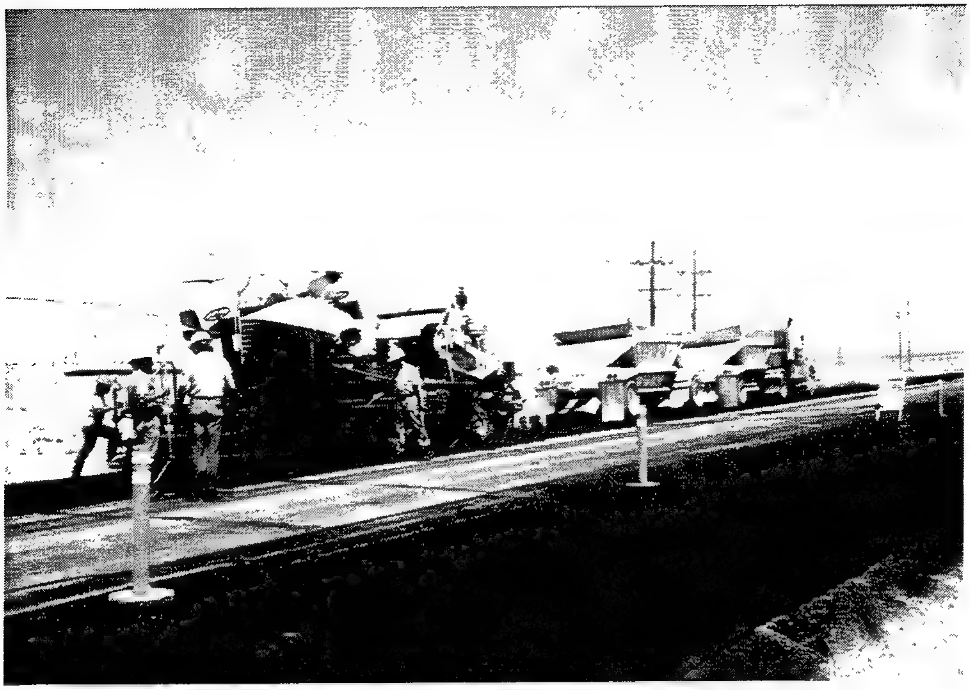


Photo 9. Belly-dump trucks placing SMA mixture in front of the paver

Photo 10. Elevator for placing windrowed SMA mixture into the paver hopper

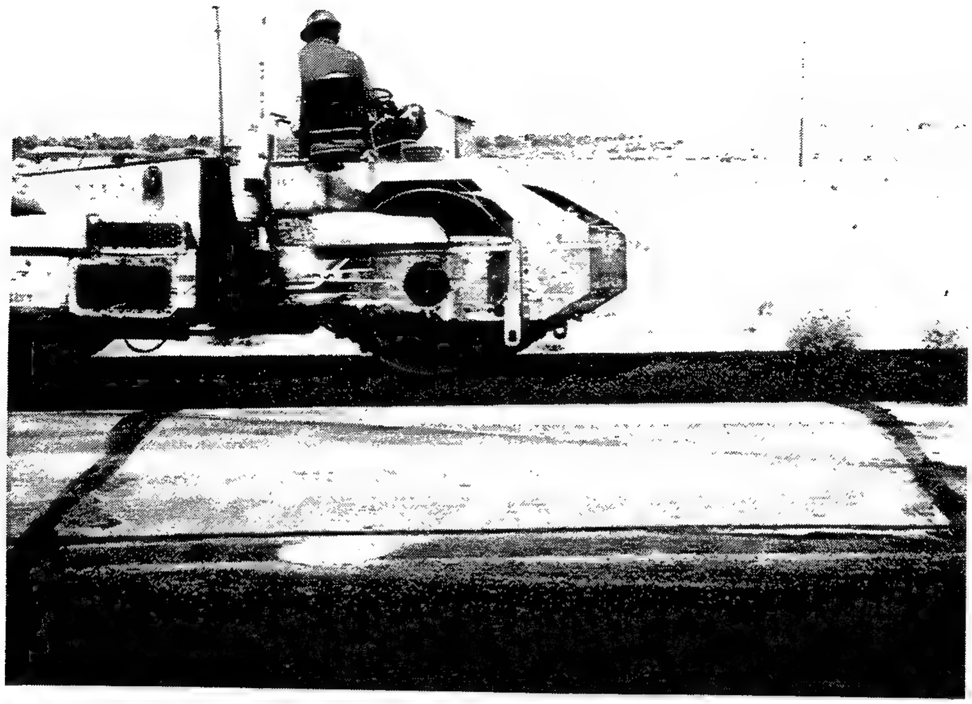


Photo 11.   Steel-wheel roller compacting SMA mixture

Photo 12.   Applying water to facilitate cooling of the SMA pavement

Photo 13. SMA pavement at the intersection of Lancaster Blvd. and Wolfe Ave. immediately after construction (looking south)

Photo 14. SMA pavement at the intersection of Lancaster Blvd. and Wolfe Ave. after 3 years of traffic (looking south, same area as shown in Photo 13)

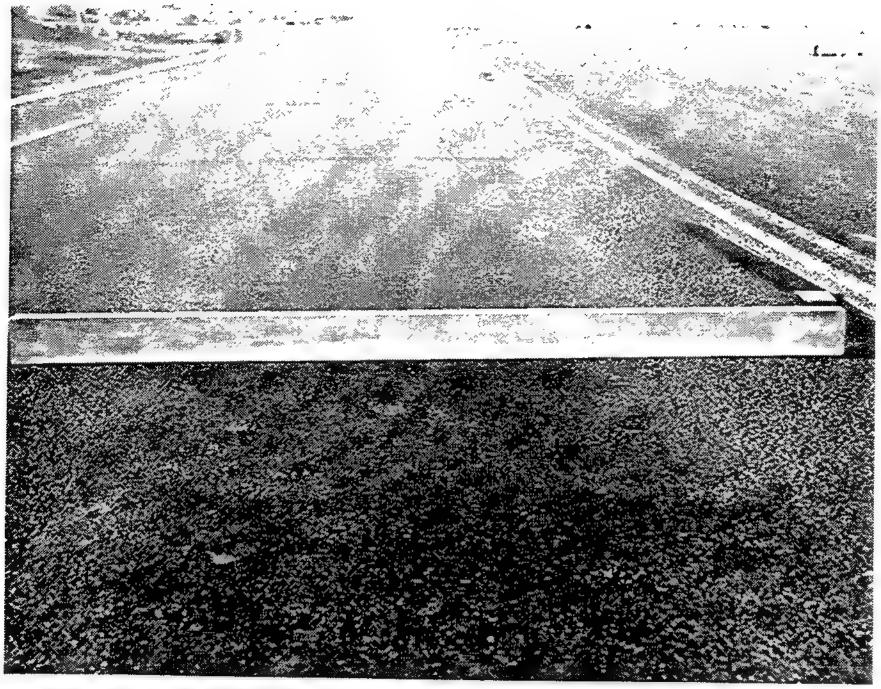


Photo 15. Typical transverse smoothness of southbound lane on Lancaster Blvd., note transverse lines in background caused by excess asphalt

Photo 16. Westbound lane of Wolfe Ave. at the intersection, note the measurable rut (approximately 0.8 cm (5/16 in.)) near the center of the straightedge in the left wheel path

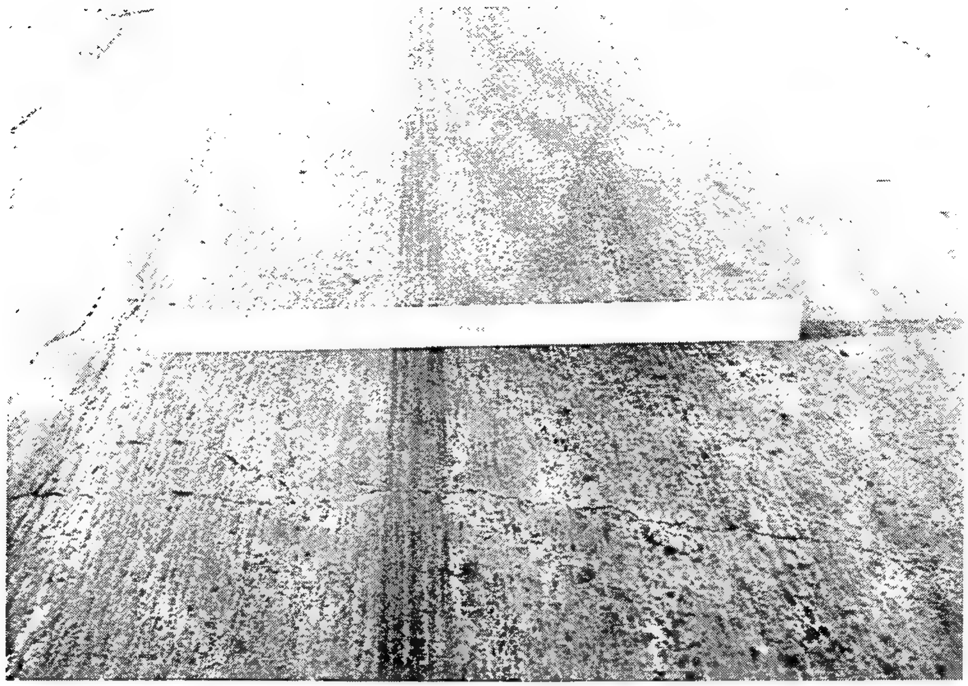


Photo 17. Surface smoothness of Lancaster Blvd. south of SMA demonstration project area

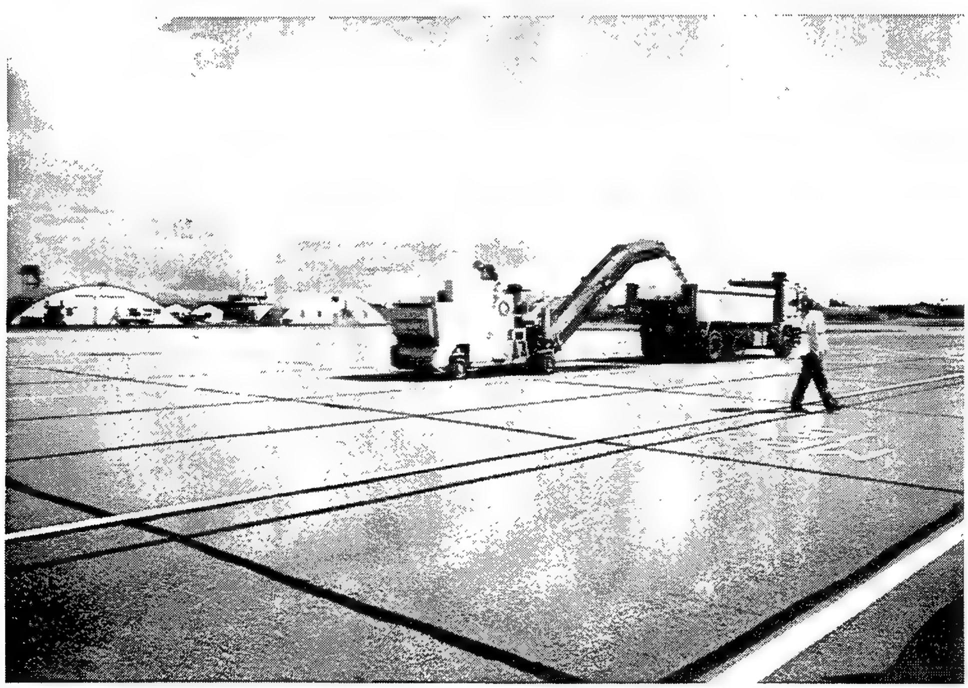


Photo 18. Cold milling of existing taxiway pavement in SMA demonstration project area (RAF Lakenheath)

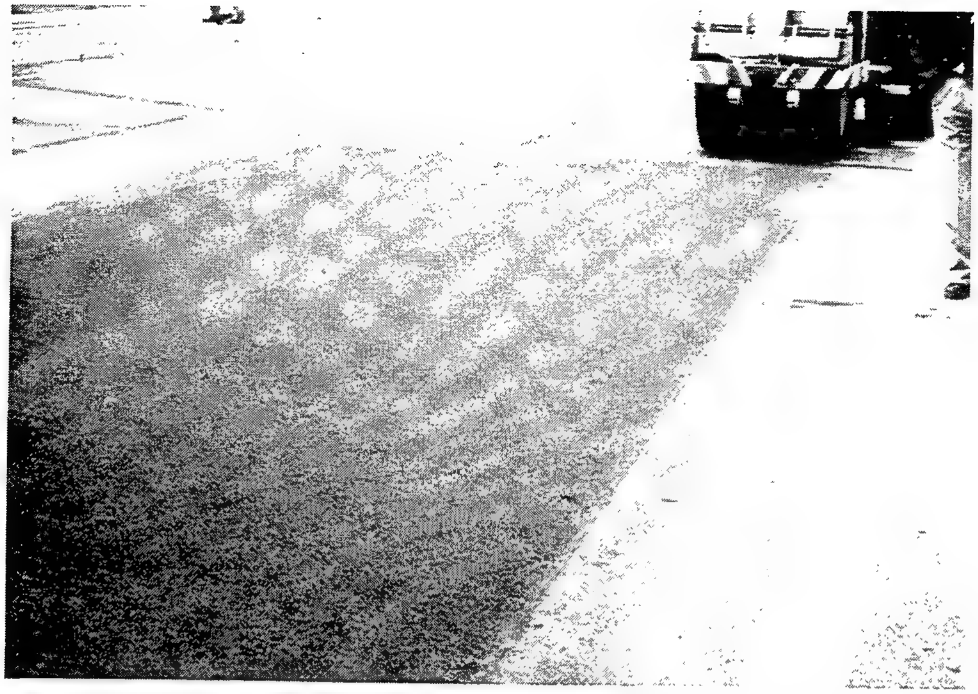
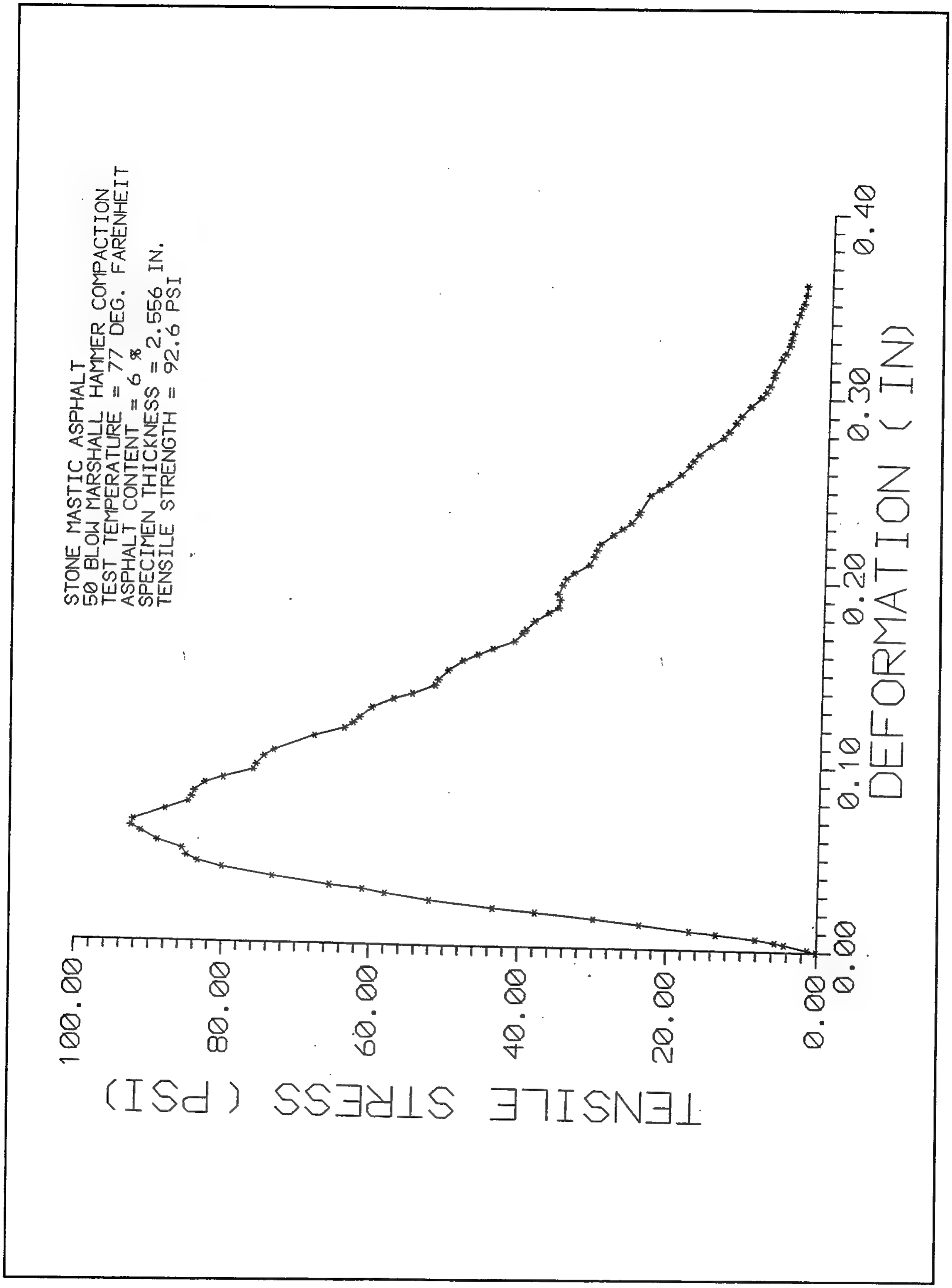
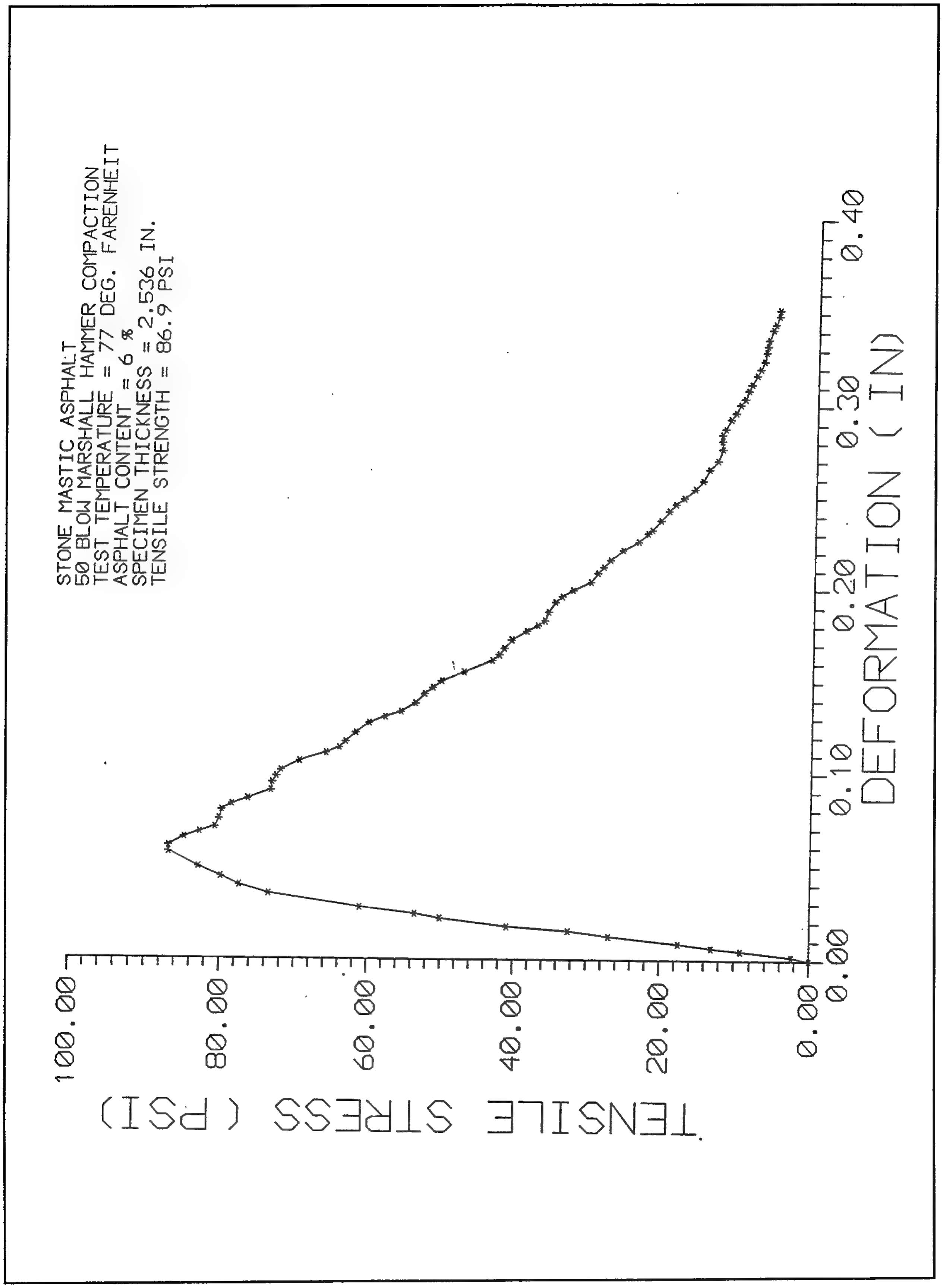


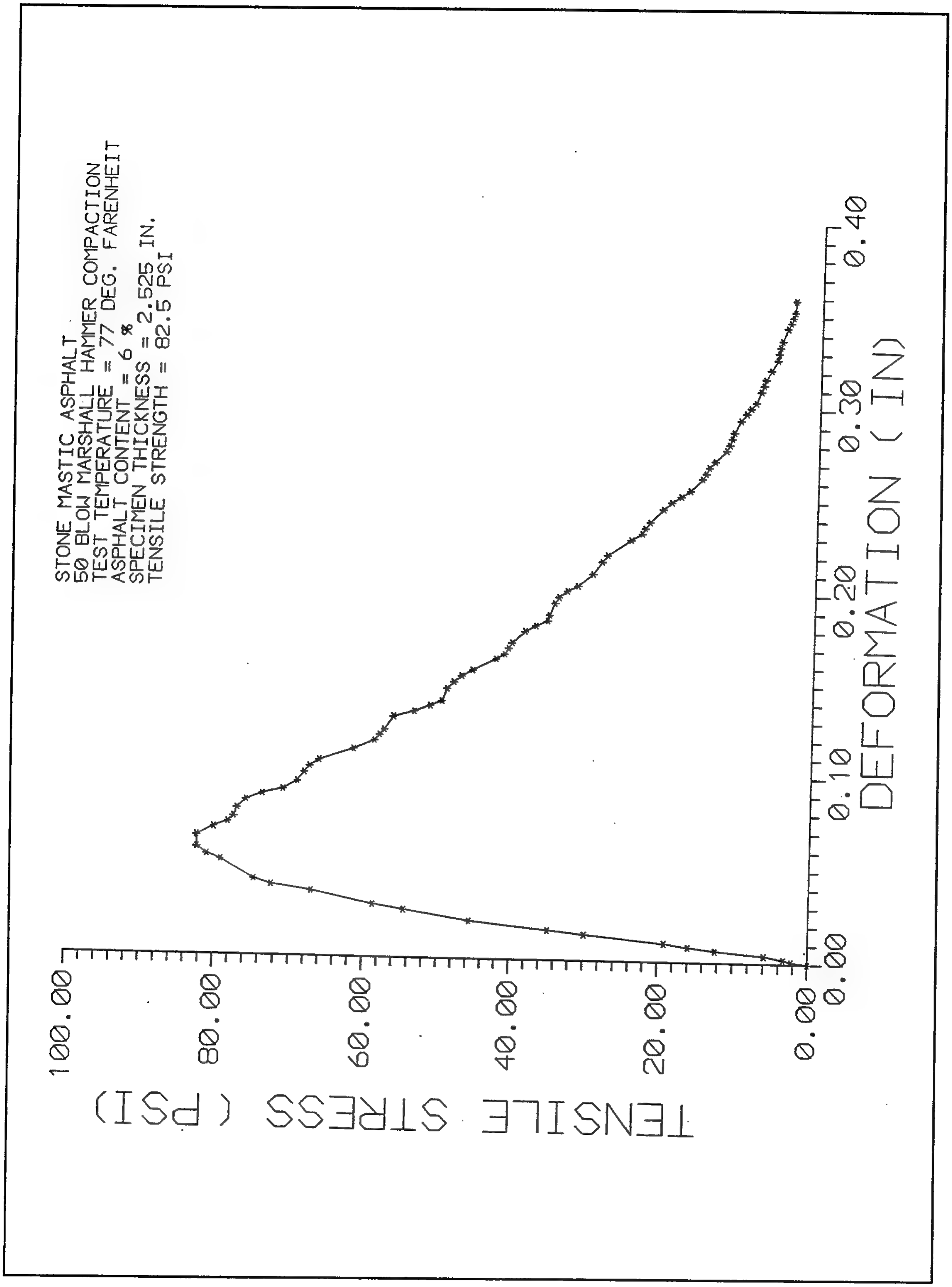
Photo 19. View of SMA on taxiway after final rolling (RAF Lakenheath)

Photo 20. Placing of SMA on taxiway with breakdown rolling, three-wheeled roller in background (RAF Lakenheath)

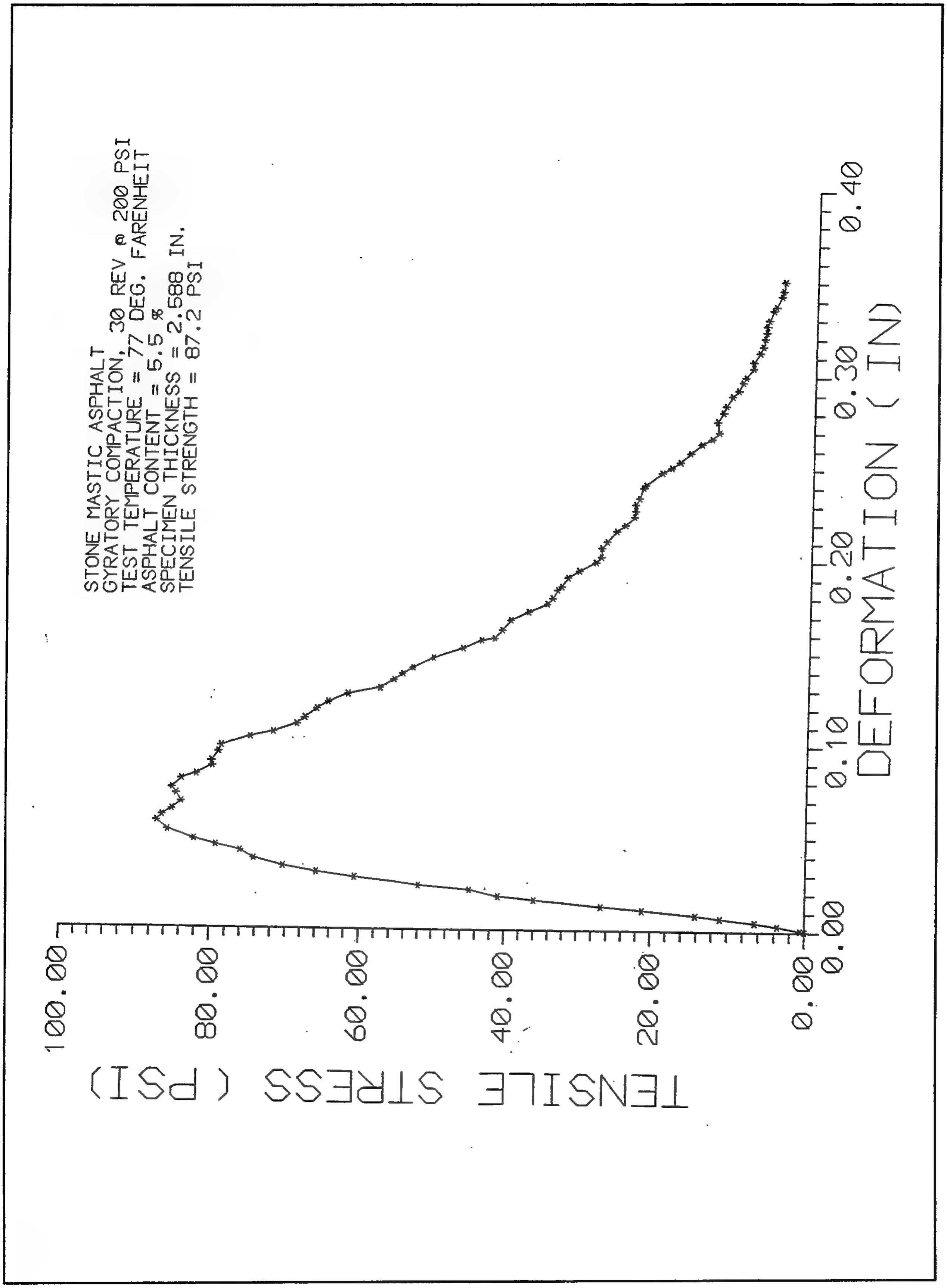
Appendix A
Indirect Tensile Tests


STONE MASTIC ASPHALT
50 BLOW MARSHALL HAMMER COMPACTION
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6 %
SPECIMEN THICKNESS = 2.556 IN.
TENSILE STRENGTH = 92.6 PSI
TENSILE STRESS (PSI)
100.00
80.00
60.00
40.00
20.00
0.00
0.00
0.10
0.20
0.30
0.40
DEFORMATION (IN)

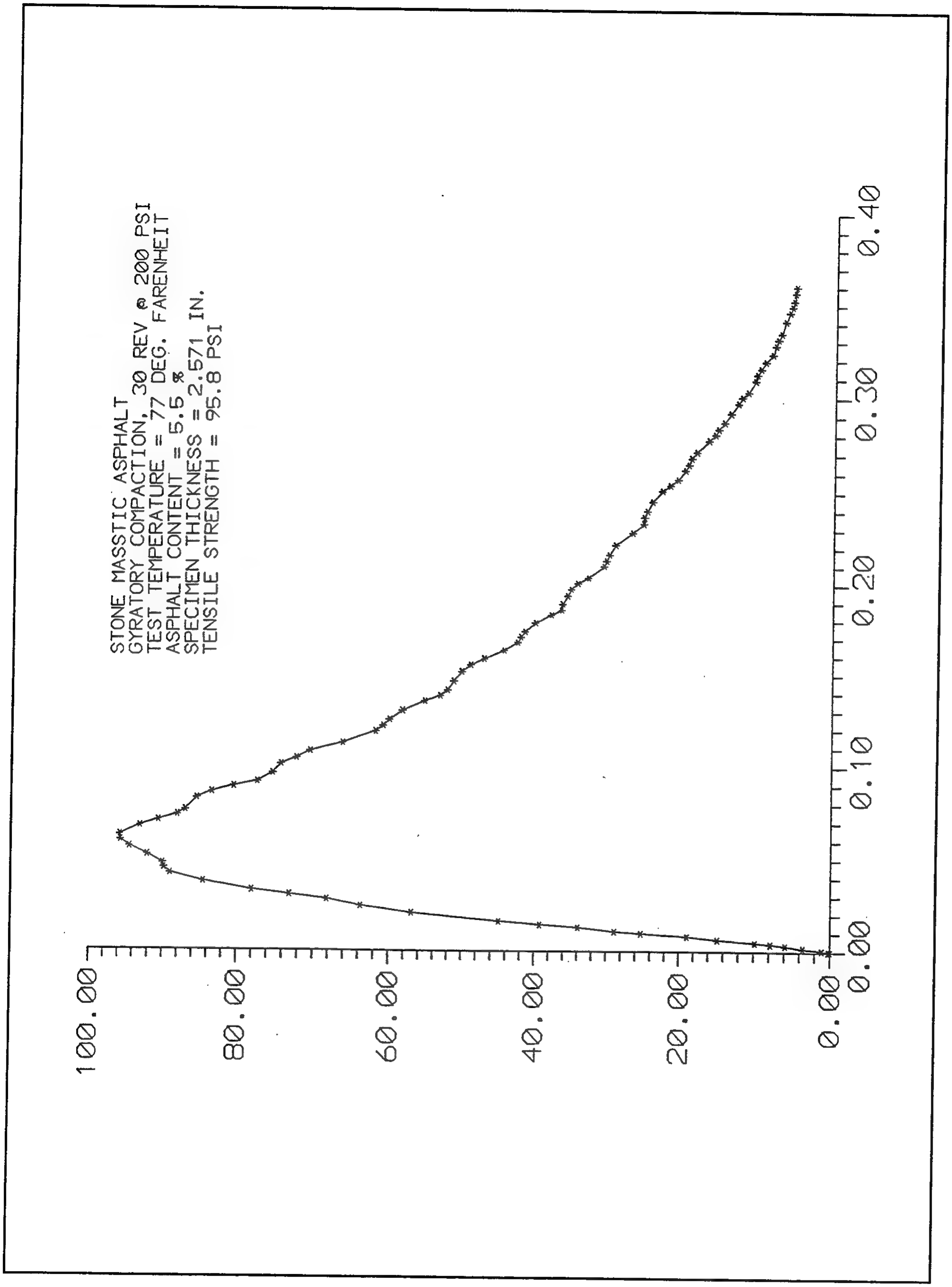

STONE MASTIC ASPHALT
50 BLOW MARSHALL HAMMER COMPACTION
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6 %
SPECIMEN THICKNESS = 2.536 IN.
TENSILE STRENGTH = 86.9 PSI
TENSILE STRESS (PSI)
100.00
80.00
60.00
40.00
20.00
0.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

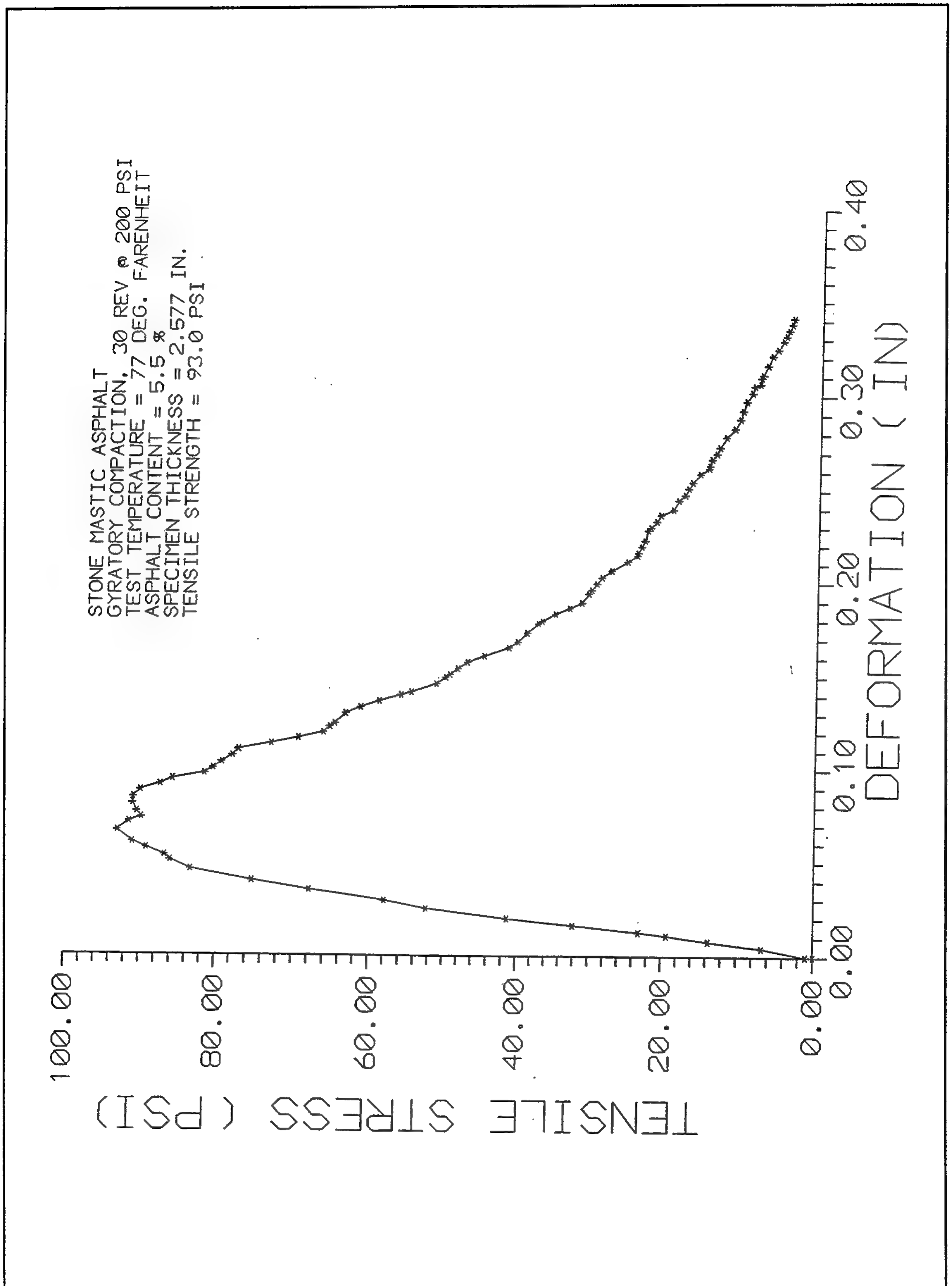


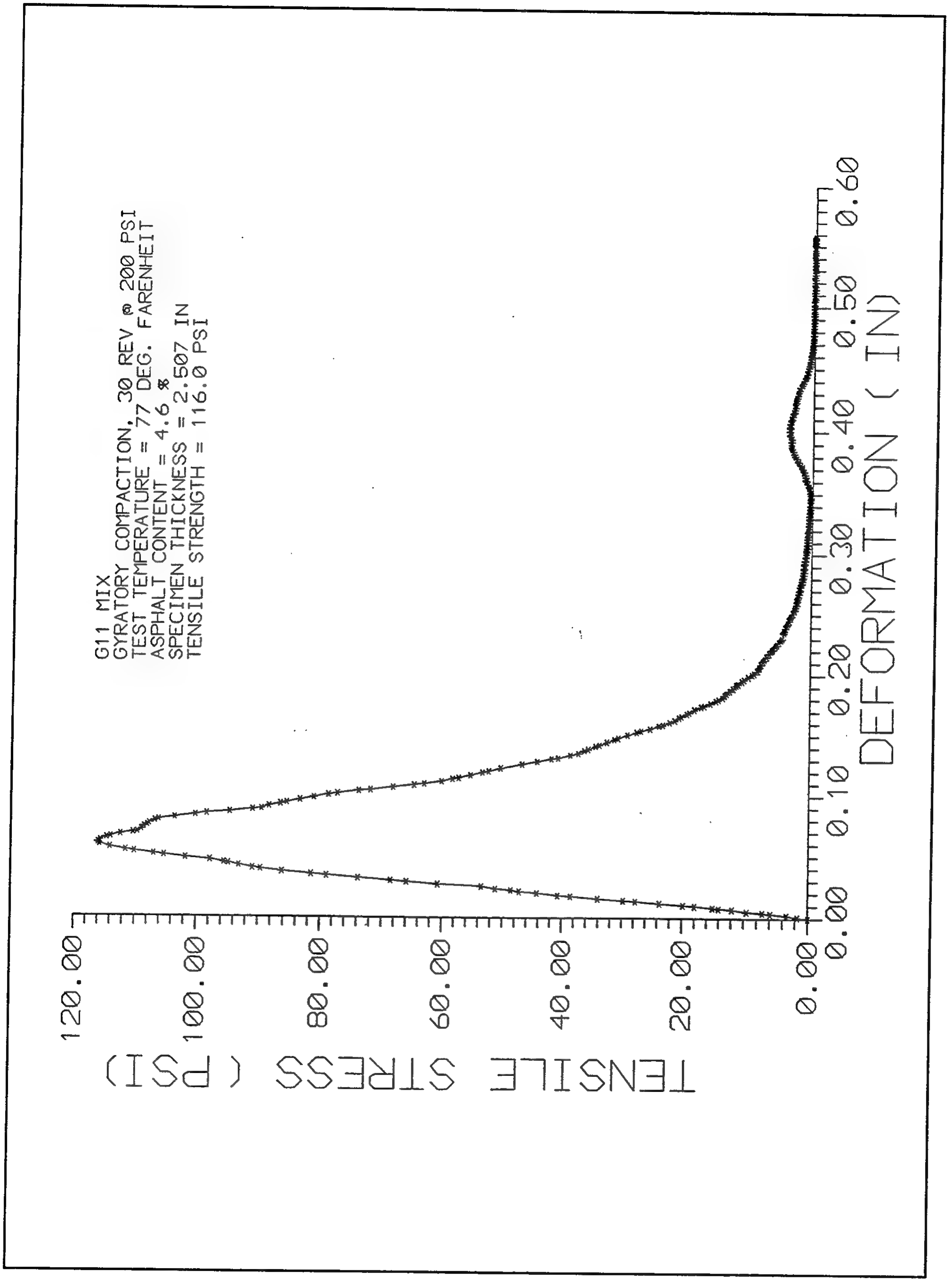
STONE MASTIC ASPHALT
50 BLOW MARSHALL HAMMER COMPACTION
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6 %
SPECIMEN THICKNESS = 2.525 IN.
TENSILE STRENGTH = 82.5 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

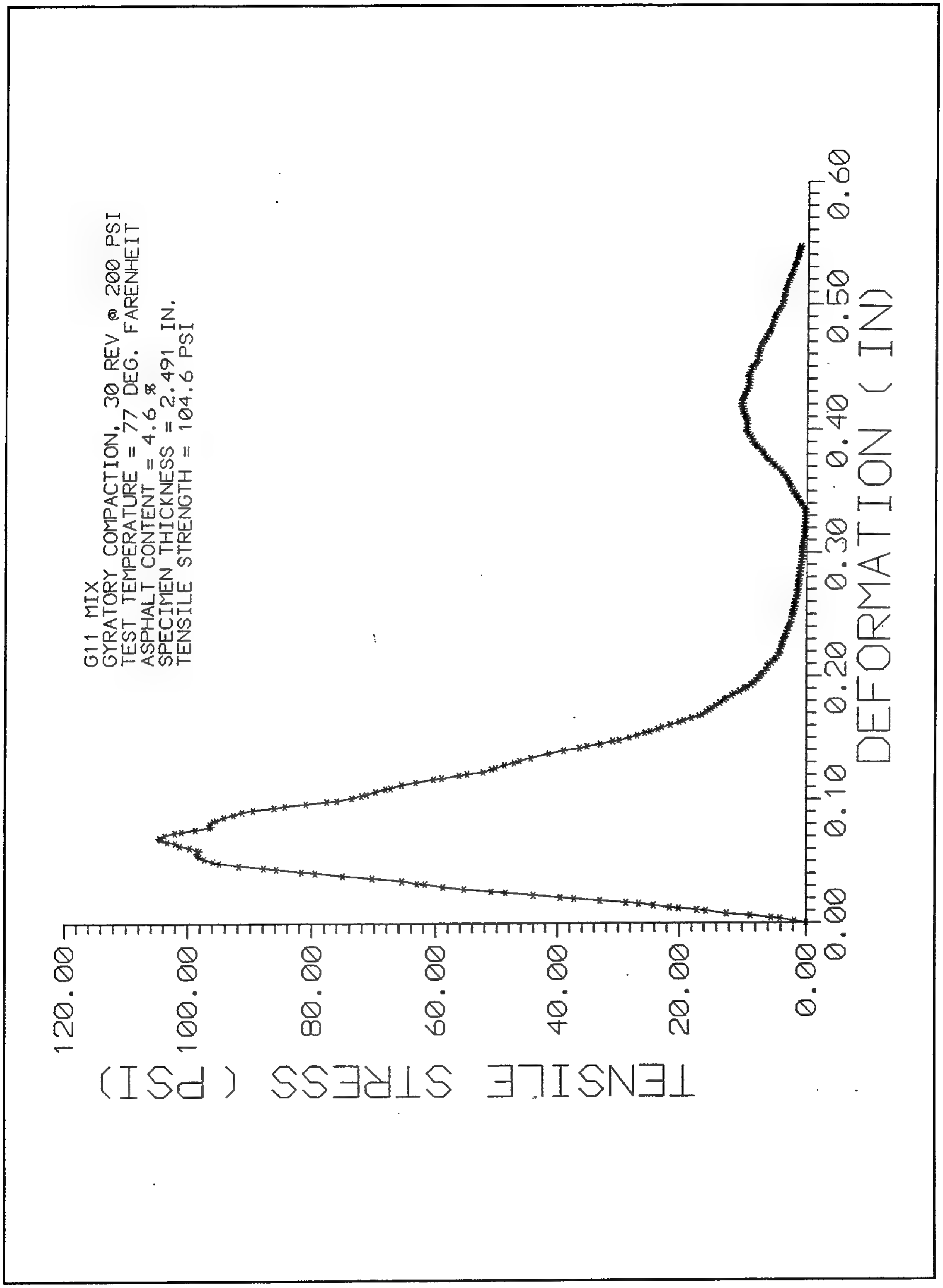


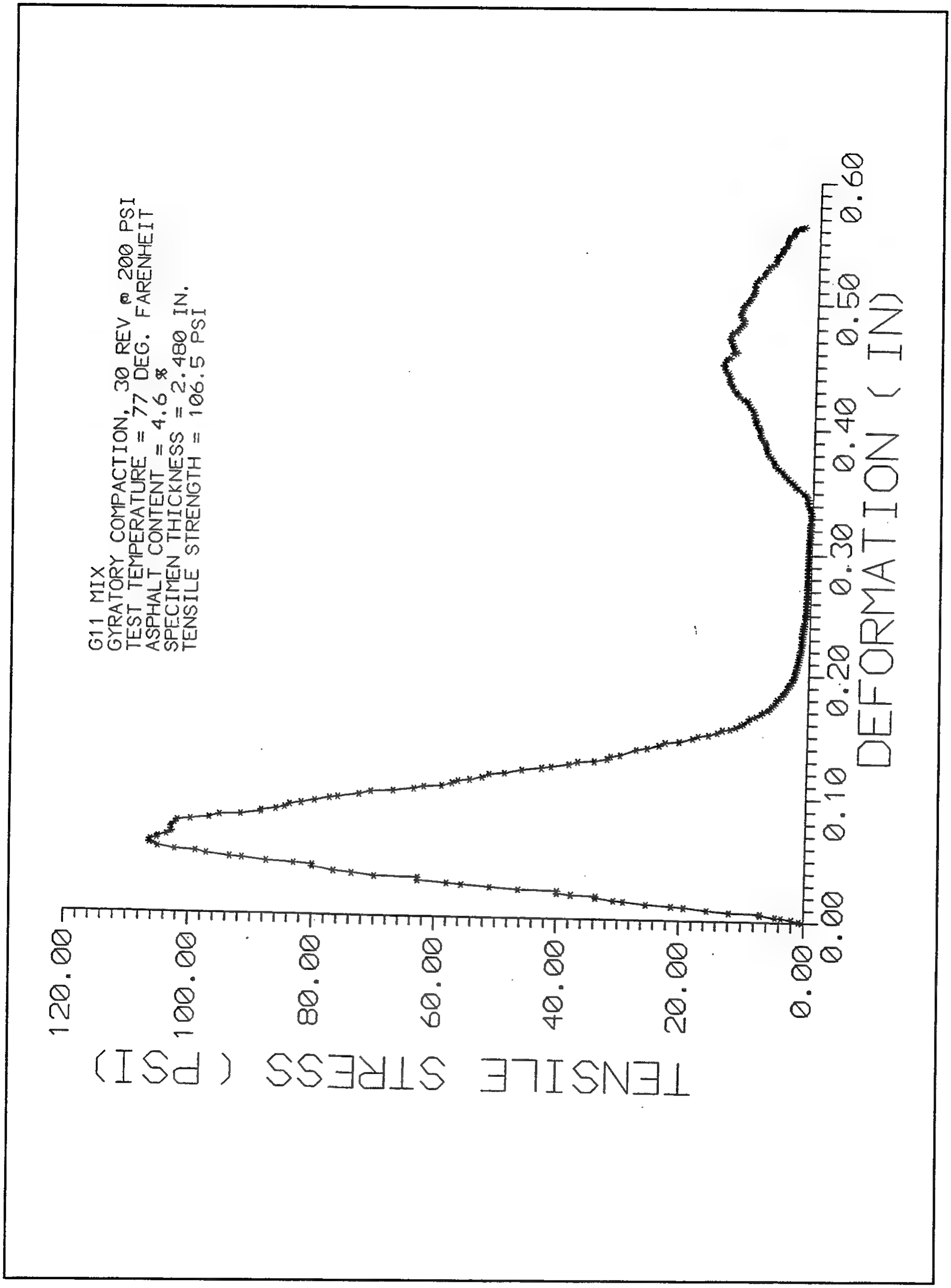
STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.5 %
SPECIMEN THICKNESS = 2.588 IN.
TENSILE STRENGTH = 87.2 PSI
TENSILE STRESS (PSI)
100.00
80.00
60.00
40.00
20.00
0.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

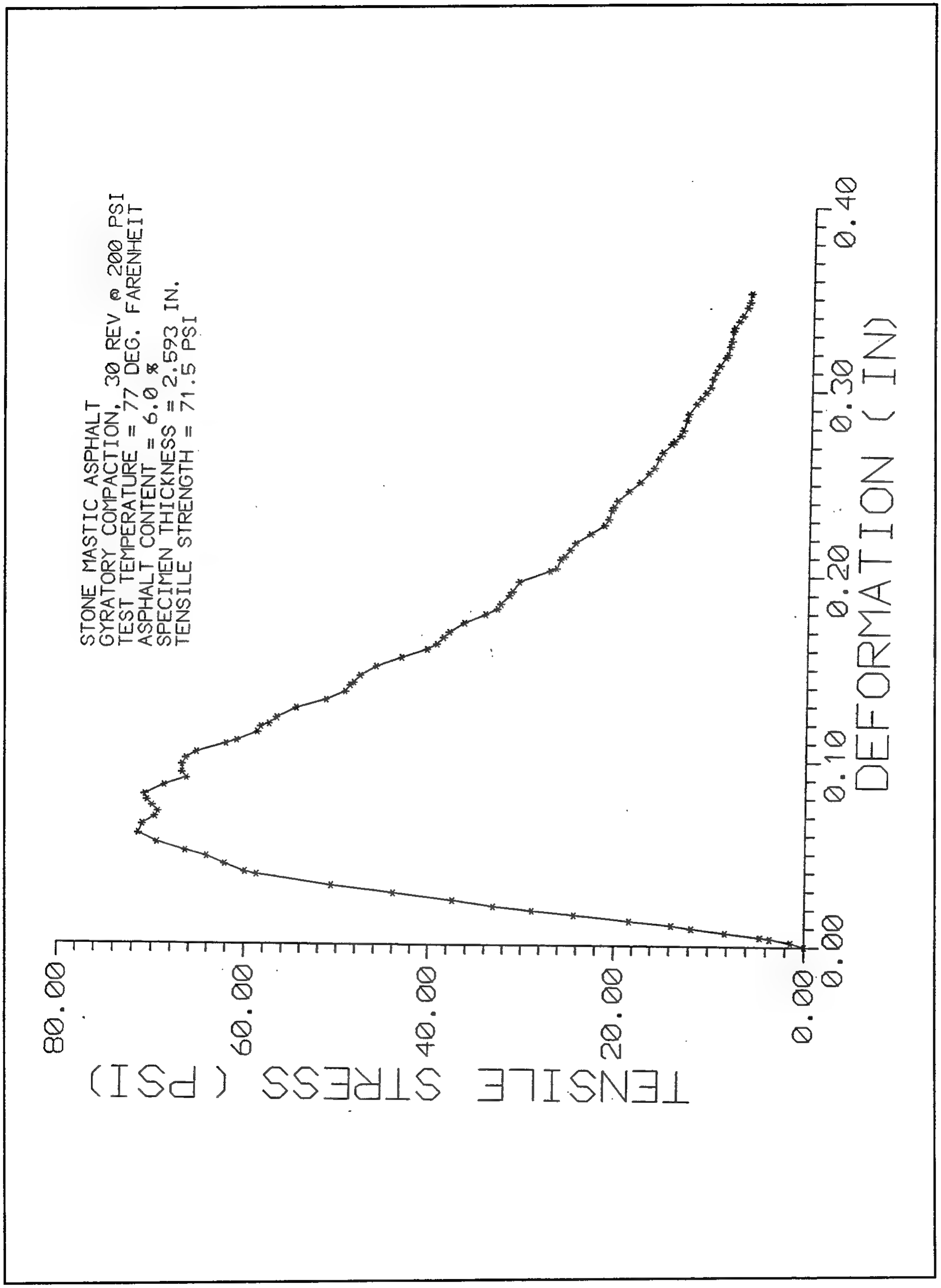

STONE MASSTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.5 %
SPECIMEN THICKNESS = 2.571 IN.
TENSILE STRENGTH = 95.8 PSI
0.00
20.00
40.00
60.00
80.00
100.00
0.00
0.10
0.20
0.30
0.40

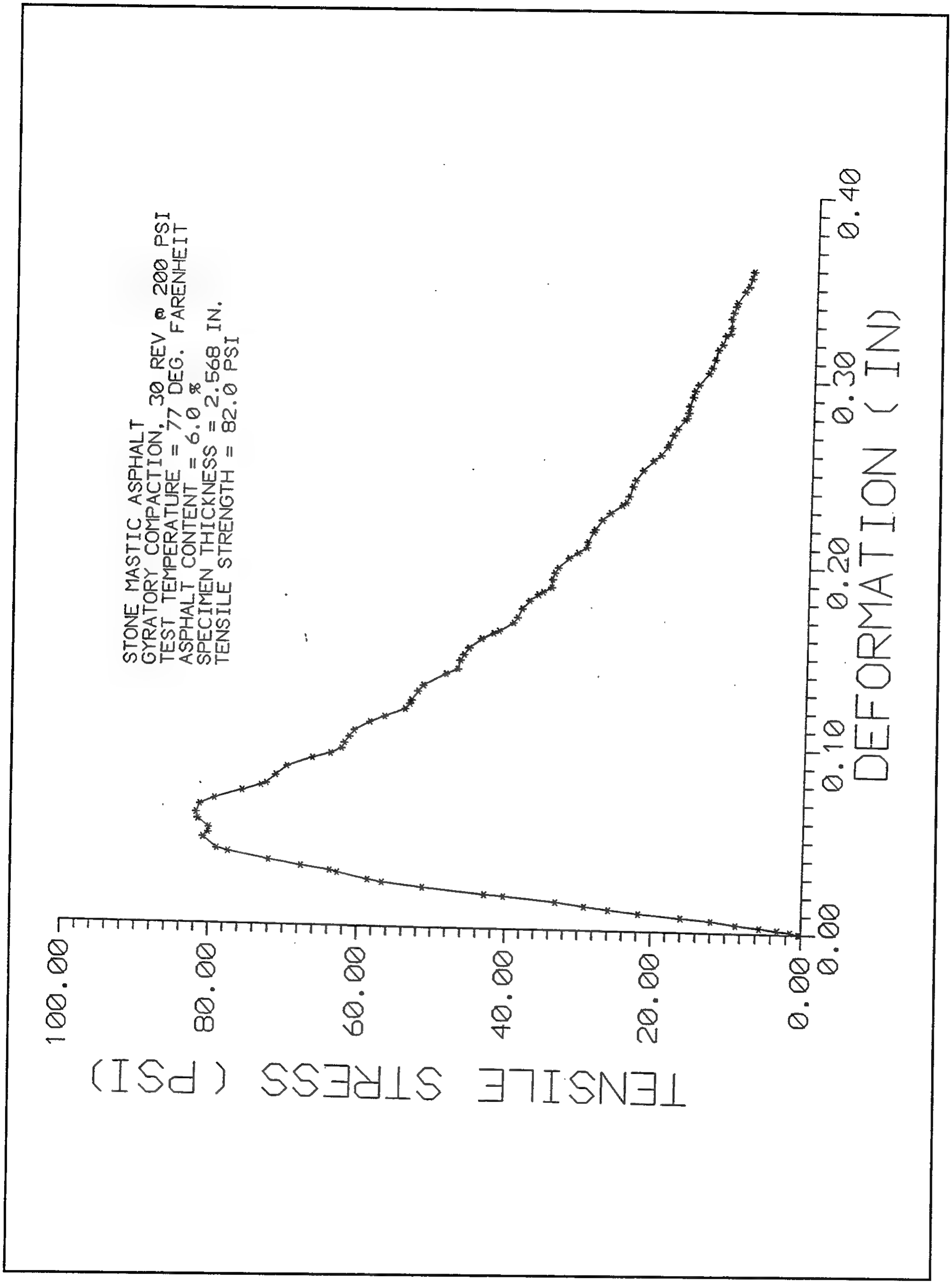

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.5 %
SPECIMEN THICKNESS = 2.577 IN.
TENSILE STRENGTH = 93.0 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40


G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 4.6 %
SPECIMEN THICKNESS = 2.507 IN
TENSILE STRENGTH = 116.0 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
120.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40
0.50
0.60

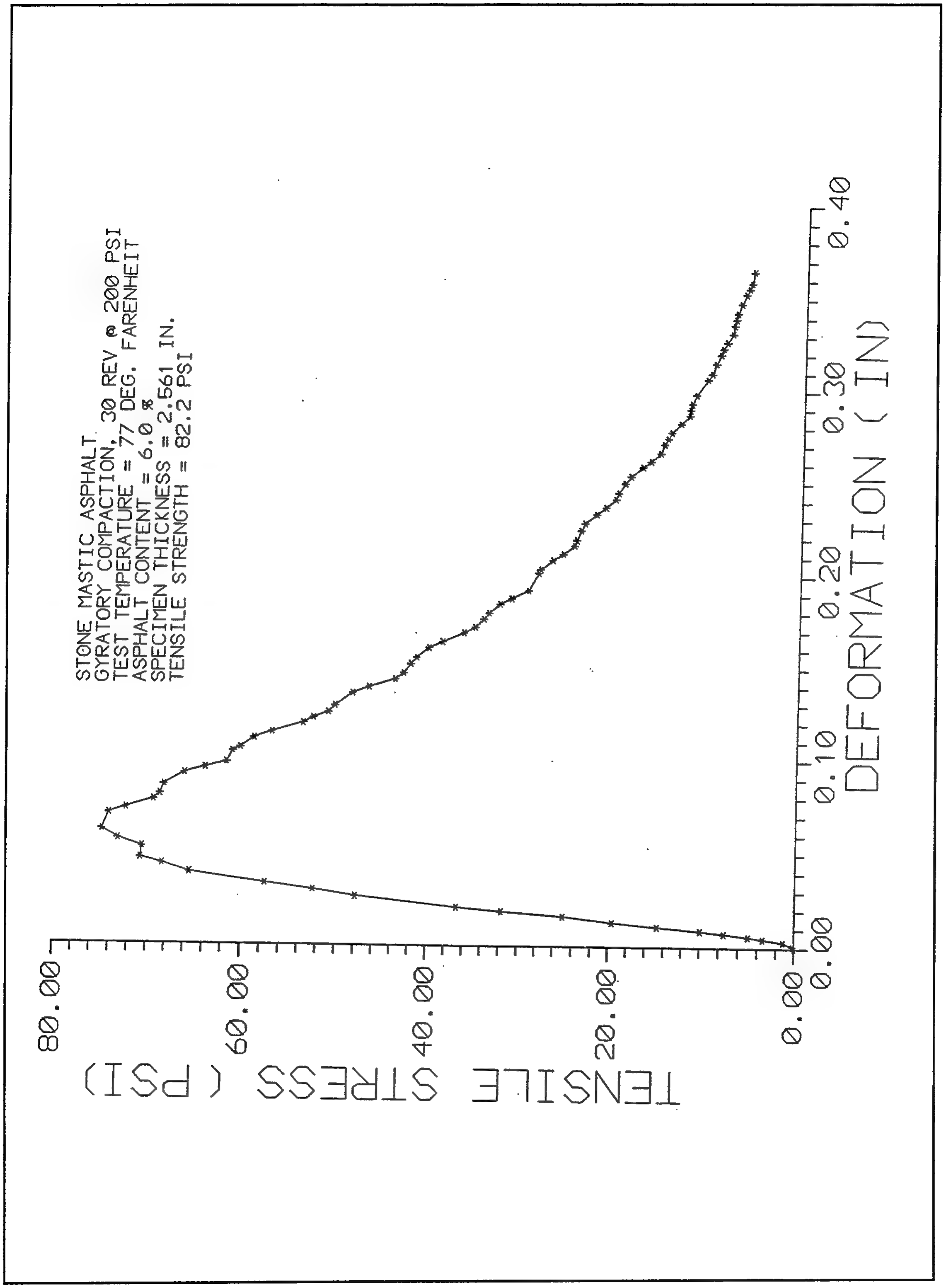

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 4.6 %
SPECIMEN THICKNESS = 2.491 IN.
TENSILE STRENGTH = 104.6 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
120.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40
0.50
0.60

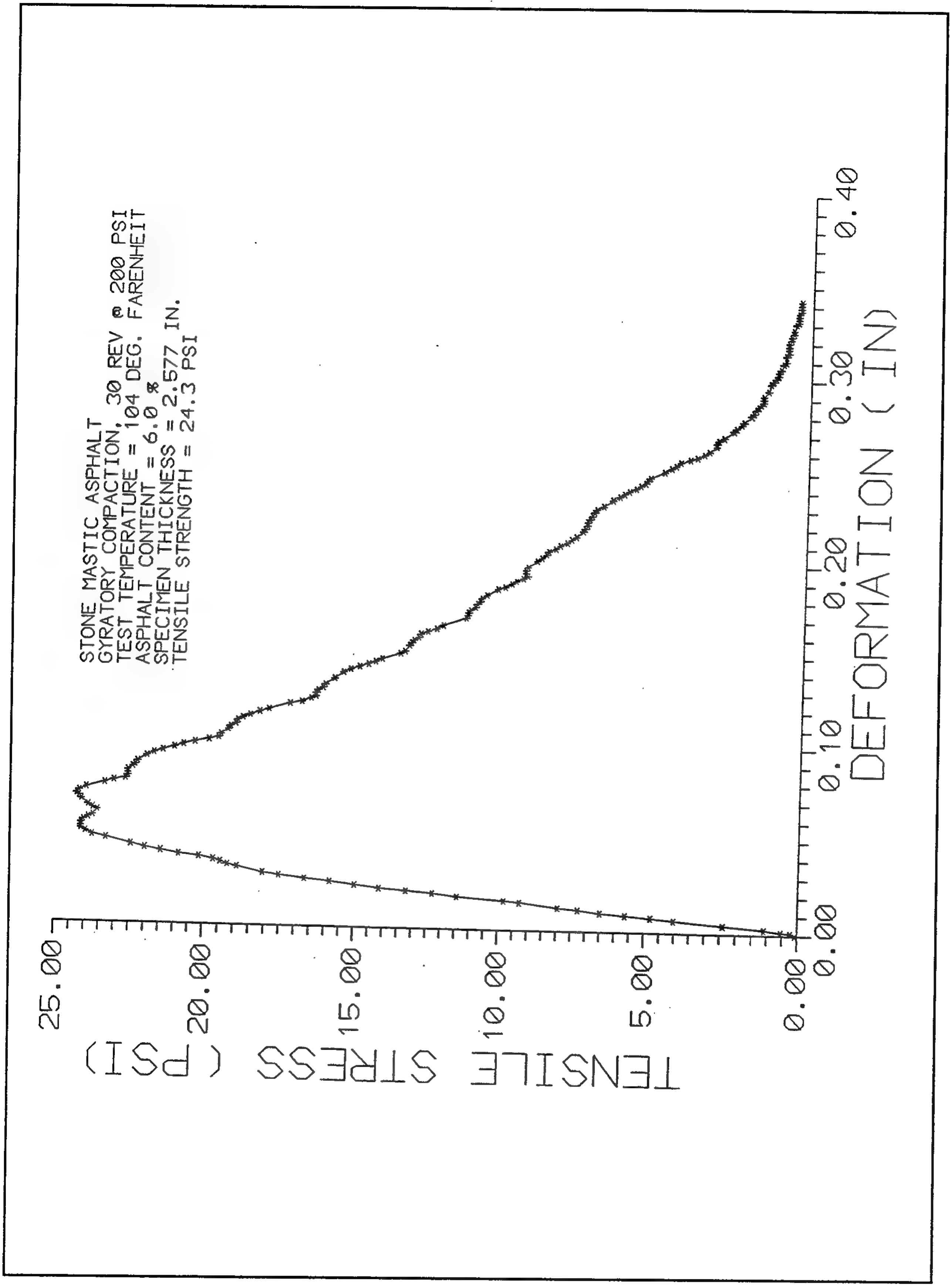

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 4.6 %
SPECIMEN THICKNESS = 2.480 IN.
TENSILE STRENGTH = 106.5 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
120.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40
0.50
0.60

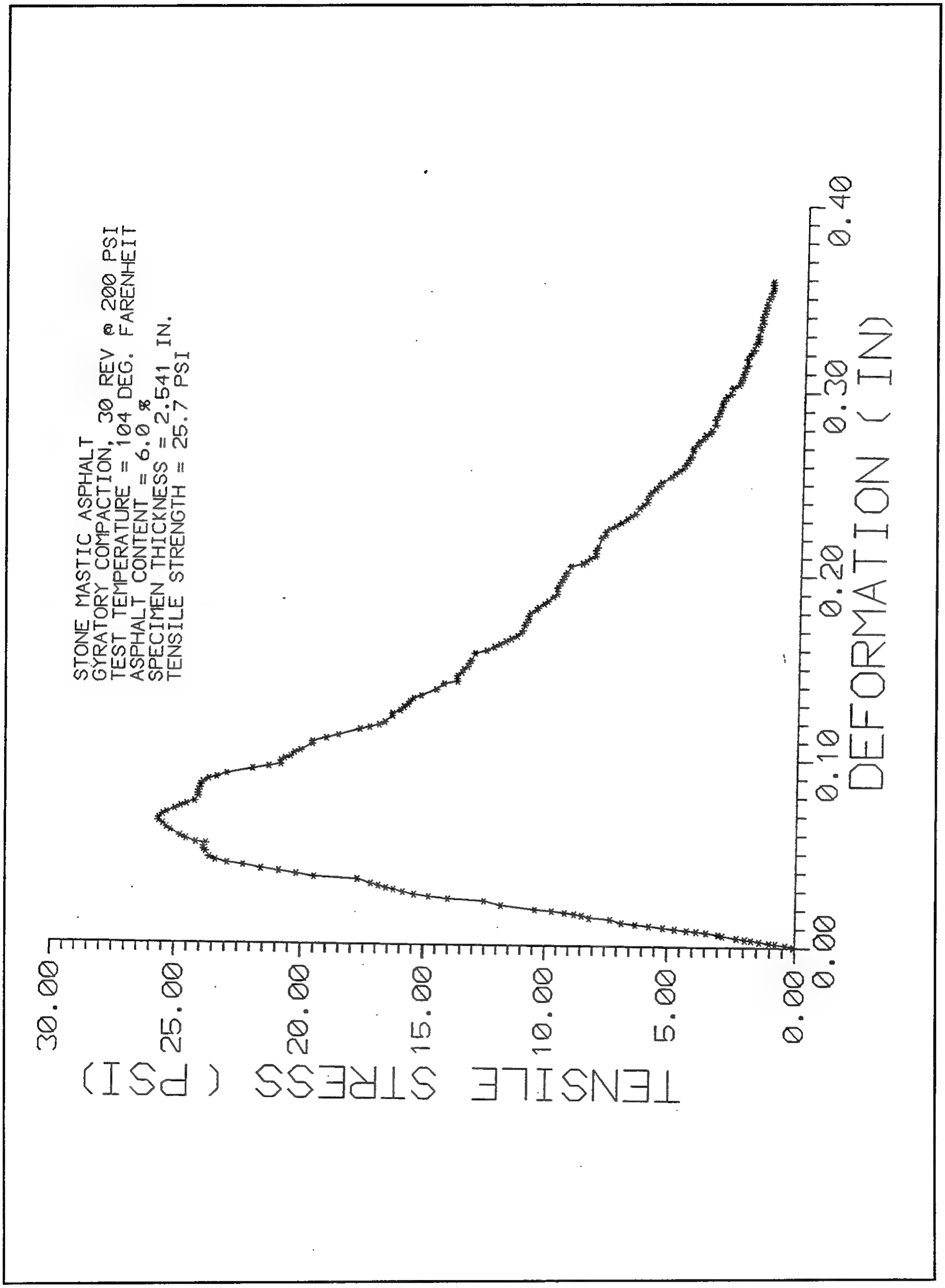

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.593 IN.
TENSILE STRENGTH = 71.5 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

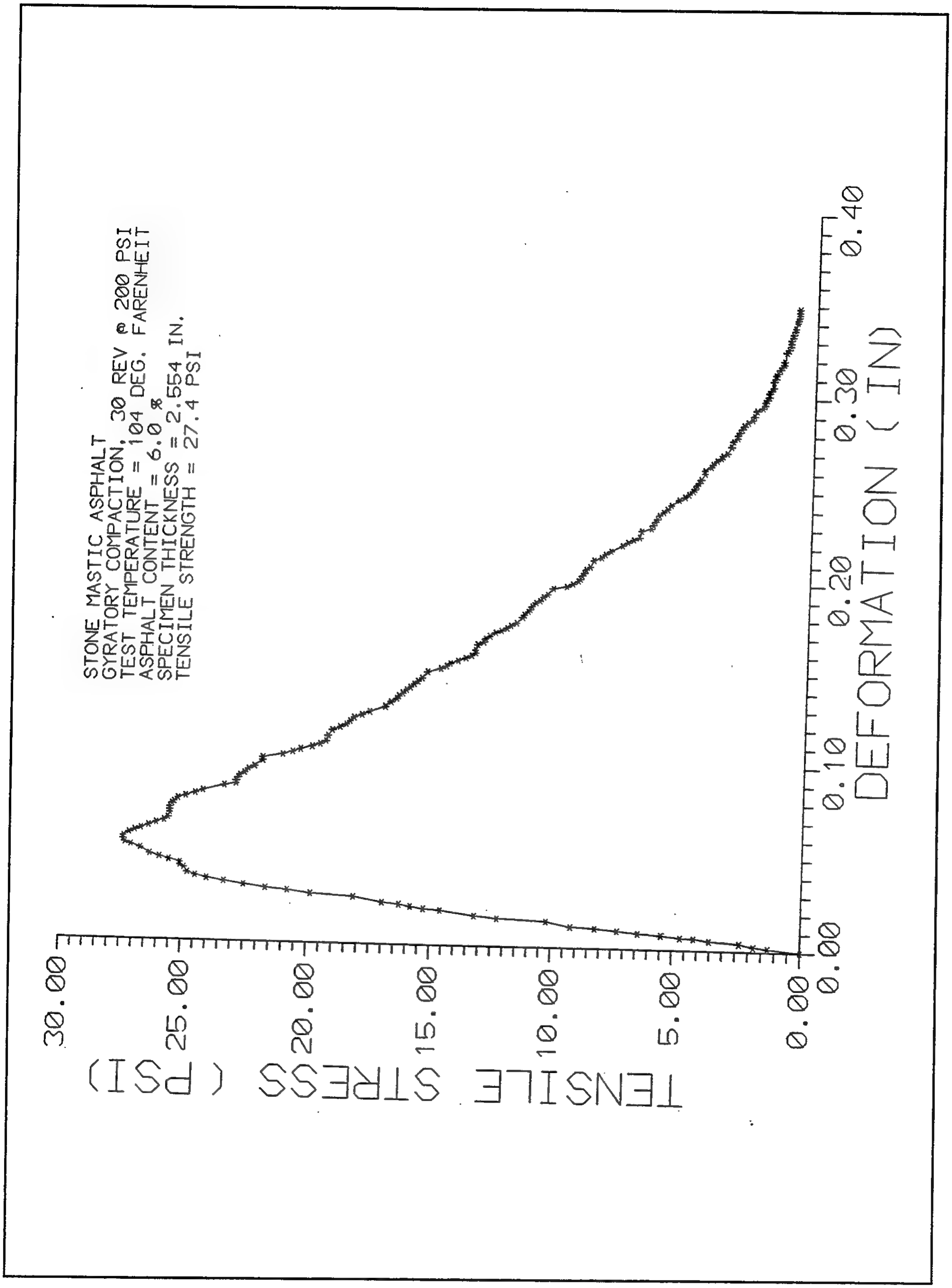


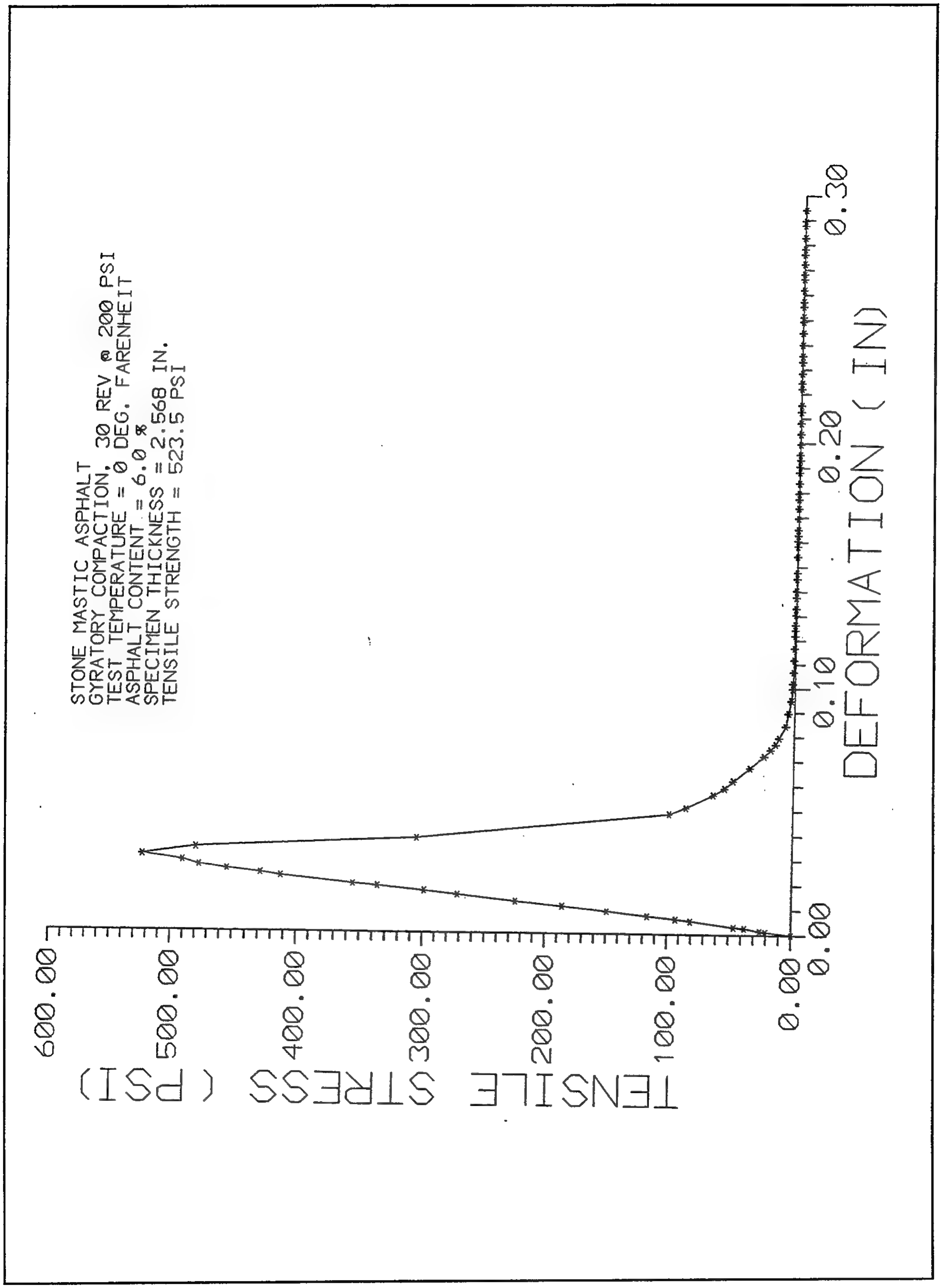
STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.568 IN.
TENSILE STRENGTH = 82.0 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

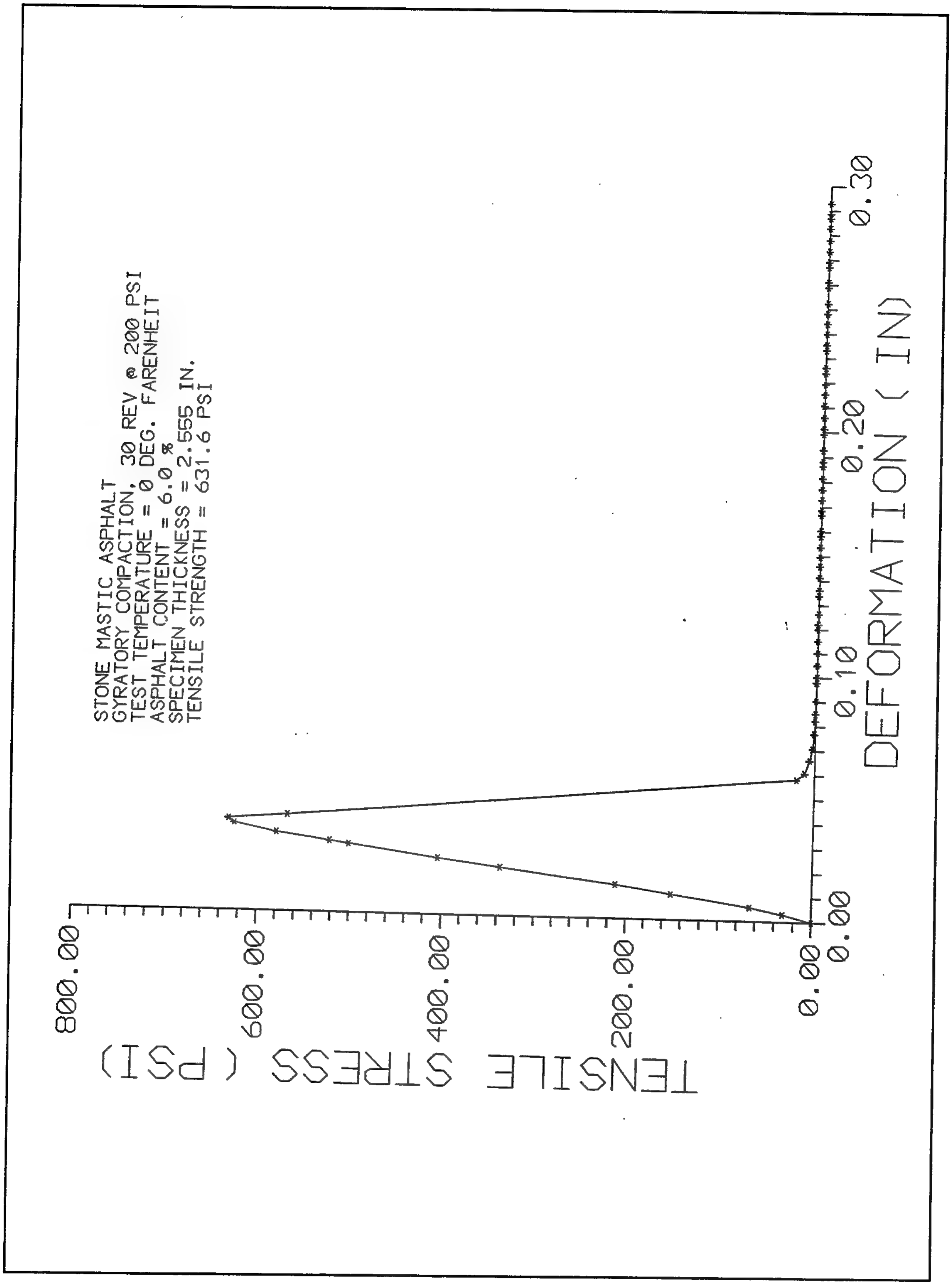

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.561 IN.
TENSILE STRENGTH = 82.2 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

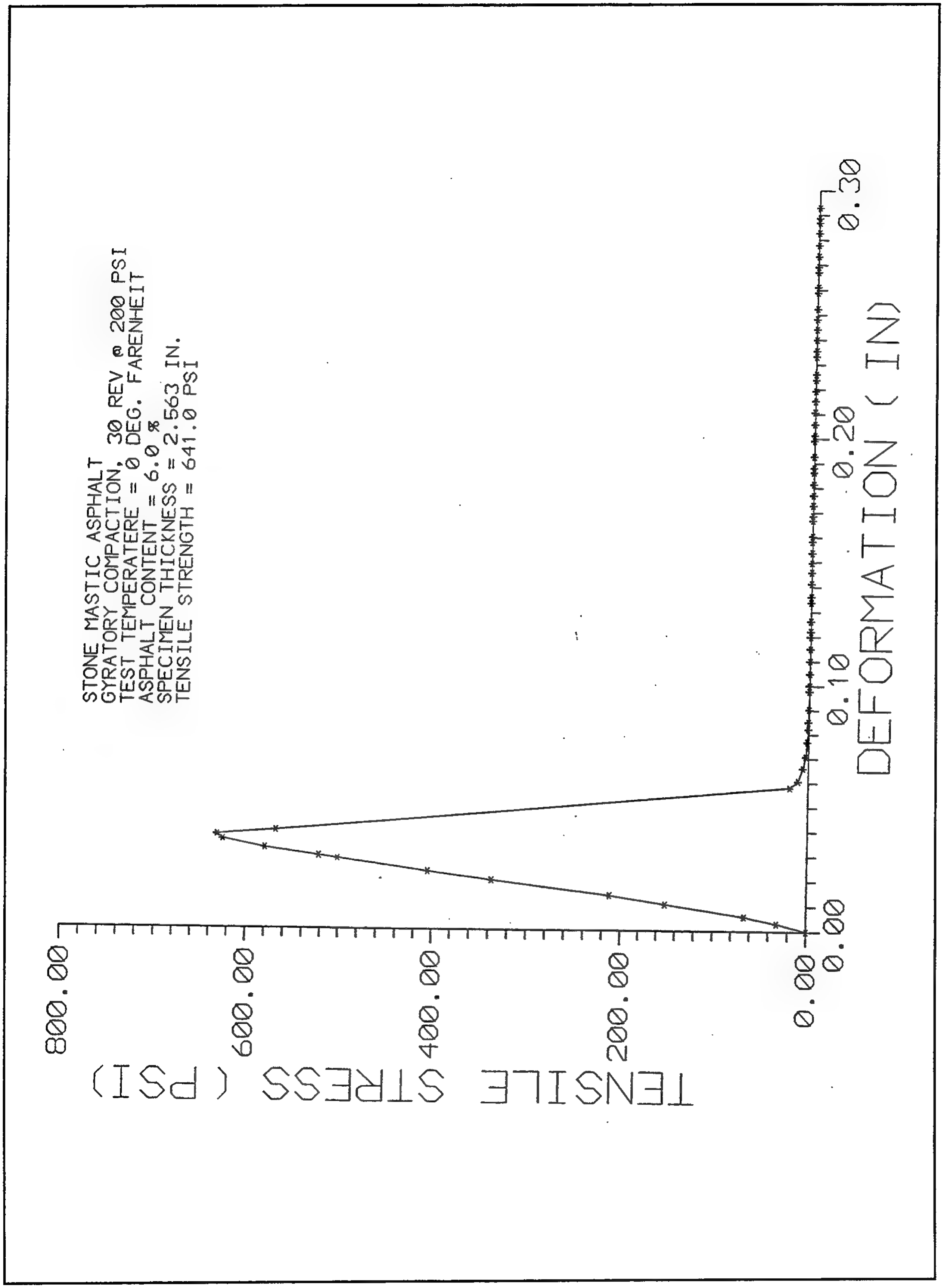

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.577 IN.
TENSILE STRENGTH = 24.3 PSI
TENSILE STRESS (PSI)
DEFORMATION (IN)
0.00
5.00
10.00
15.00
20.00
25.00
0.00
0.10
0.20
0.30
0.40

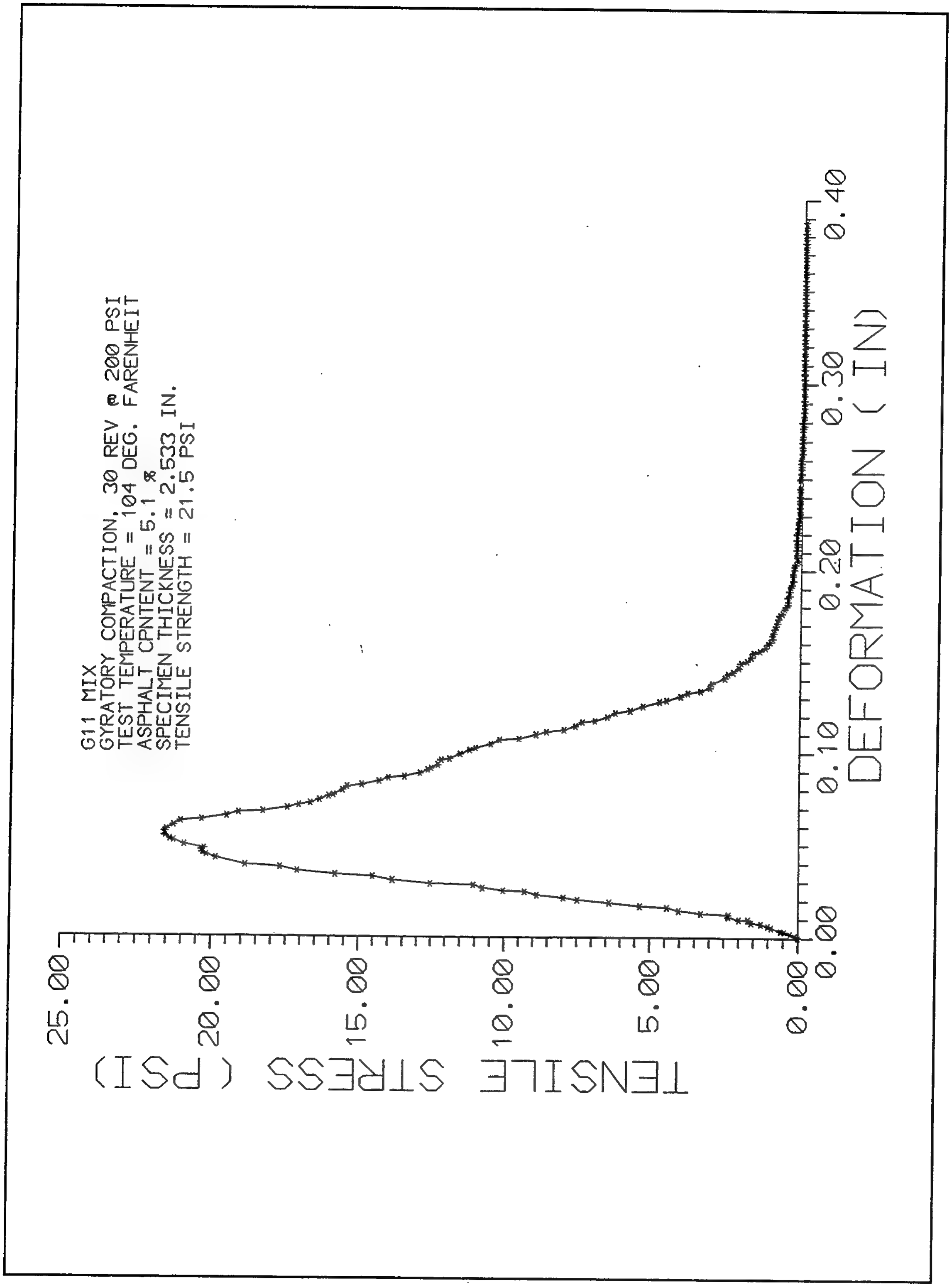

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.541 IN.
TENSILE STRENGTH = 25.7 PSI
TENSILE STRESS (PSI)
0.00
5.00
10.00
15.00
20.00
25.00
30.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

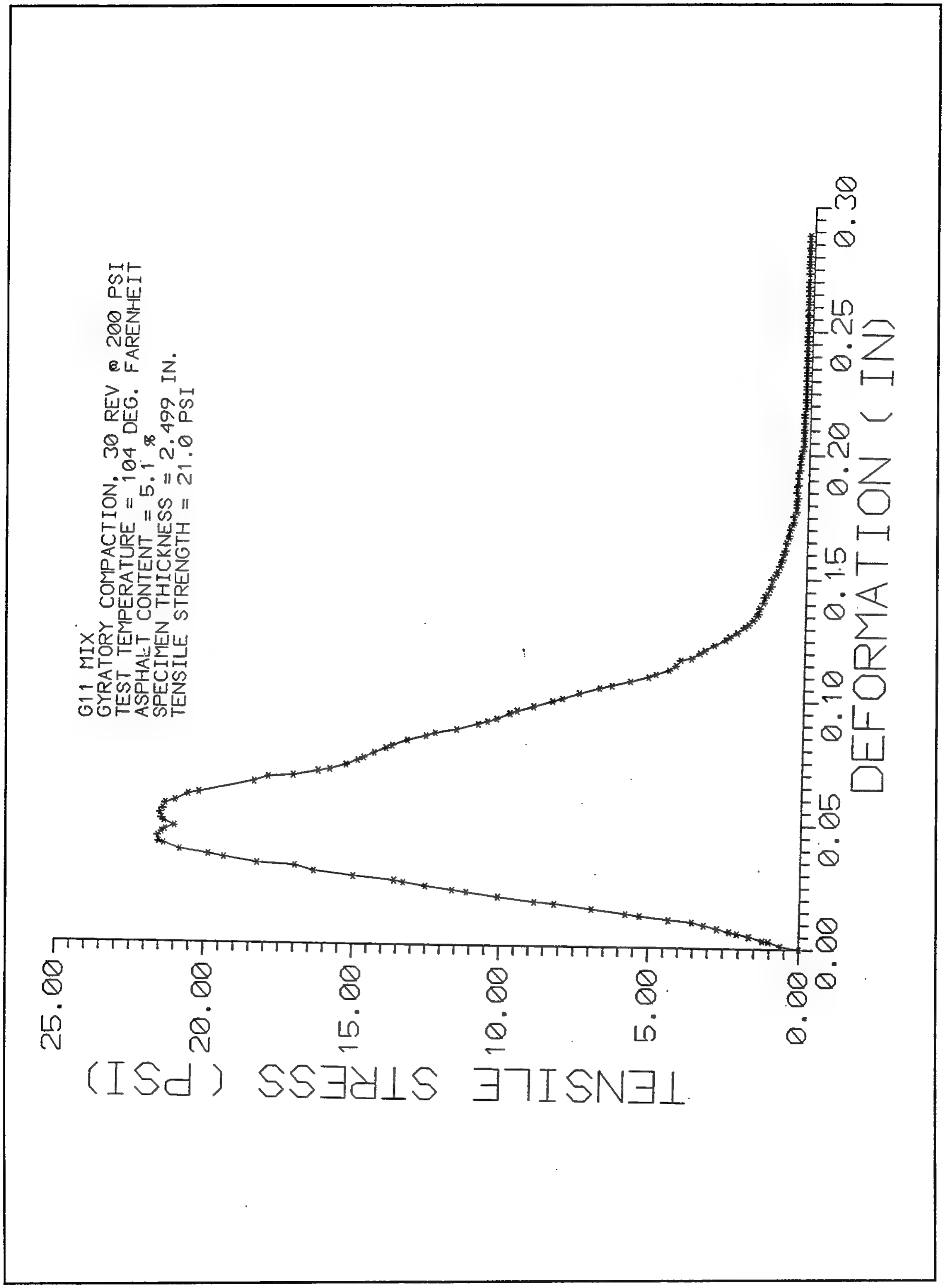

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.554 IN.
TENSILE STRENGTH = 27.4 PSI
TENSILE STRESS (PSI)
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
0.10
0.20
0.30
0.40
DEFORMATION (IN)

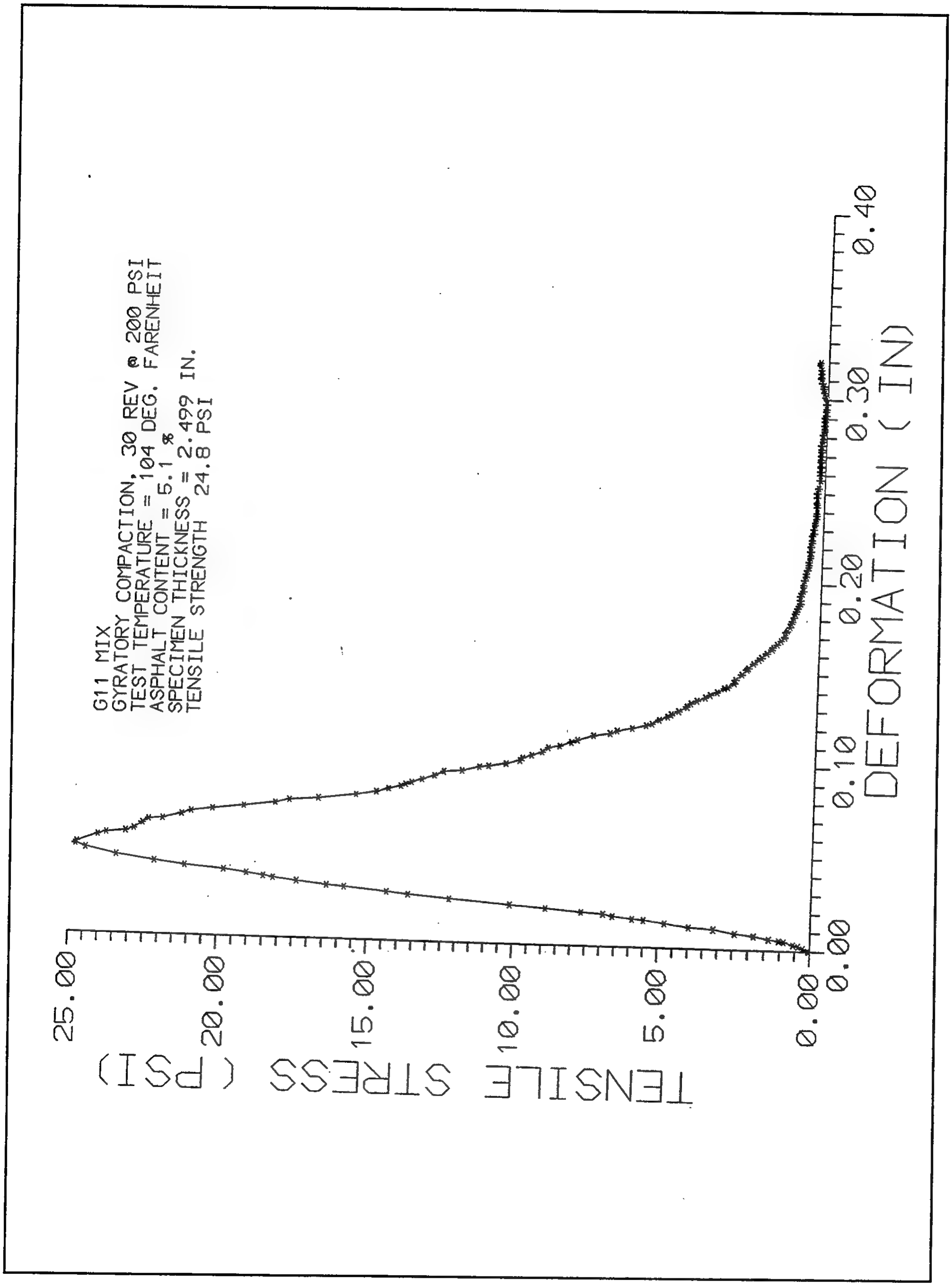

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 0 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.568 IN.
TENSILE STRENGTH = 523.5 PSI
TENSILE STRESS (PSI)
0.00
100.00
200.00
300.00
400.00
500.00
600.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30

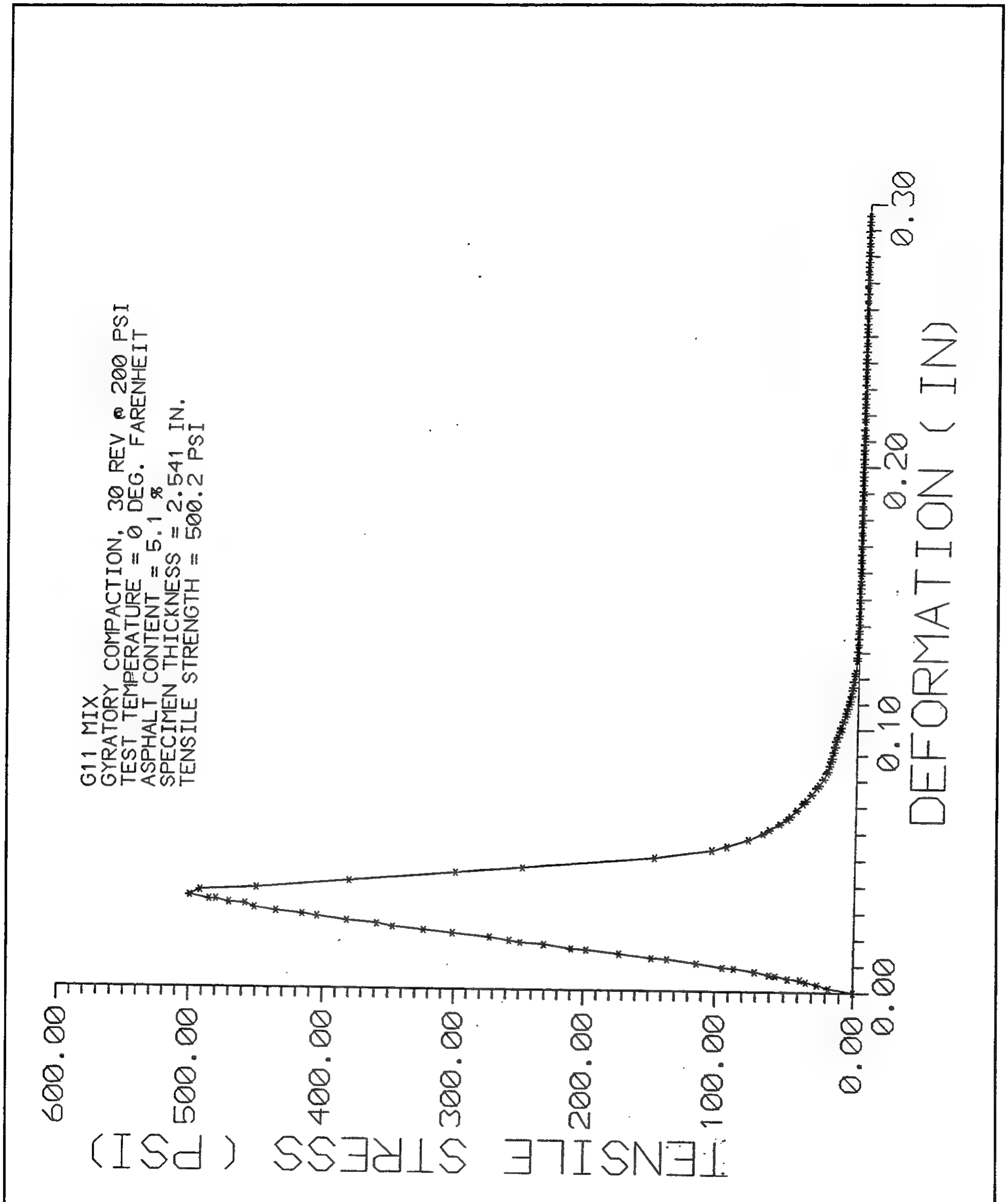

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 0 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.555 IN.
TENSILE STRENGTH = 631.6 PSI
TENSILE STRESS (PSI)
800.00
600.00
400.00
200.00
0.00
0.00
0.10
0.20
0.30
DEFORMATION (IN)

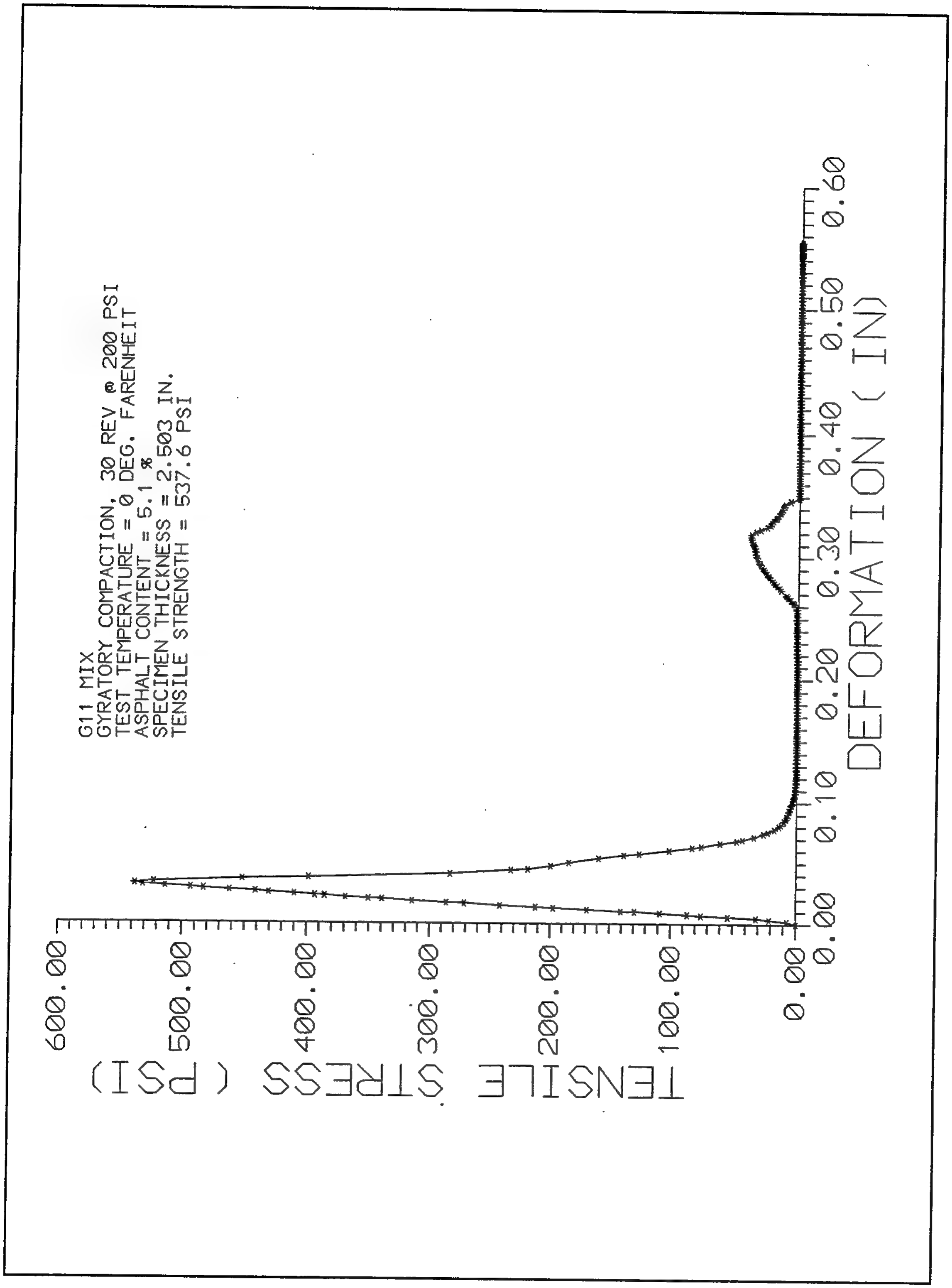

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATERE = 0 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
SPECIMEN THICKNESS = 2.563 IN.
TENSILE STRENGTH = 641.0 PSI
TENSILE STRESS (PSI)
0.00
200.00
400.00
600.00
800.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30

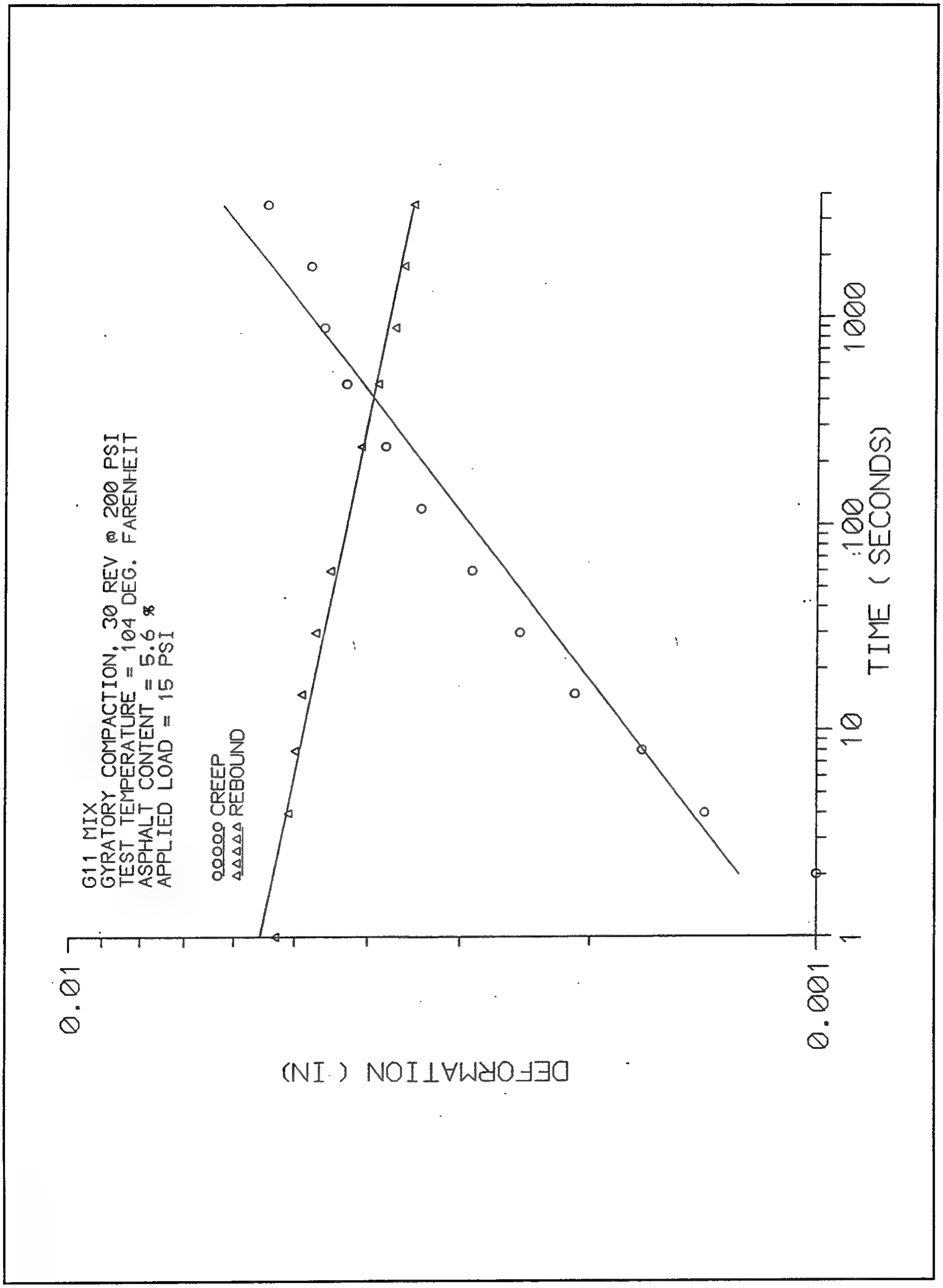

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CPNTENT = 5.1 %
SPECIMEN THICKNESS = 2.533 IN.
TENSILE STRENGTH = 21.5 PSI
TENSILE STRESS (PSI)
0.00
5.00
10.00
15.00
20.00
25.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

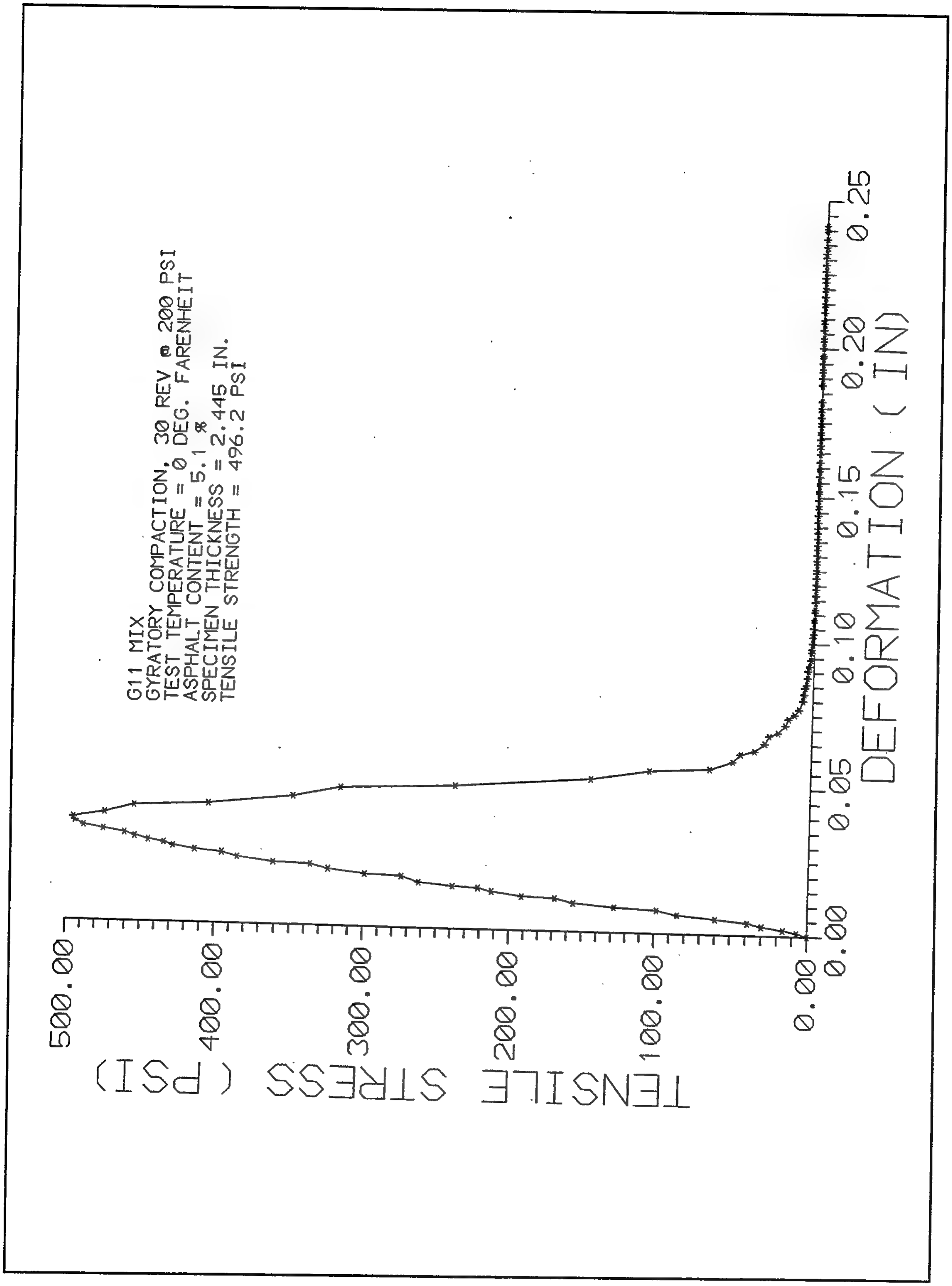

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
SPECIMEN THICKNESS = 2.499 IN.
TENSILE STRENGTH = 21.0 PSI
TENSILE STRESS (PSI)
0.00
5.00
10.00
15.00
20.00
25.00
DEFORMATION (IN)
0.00
0.05
0.10
0.15
0.20
0.25
0.30

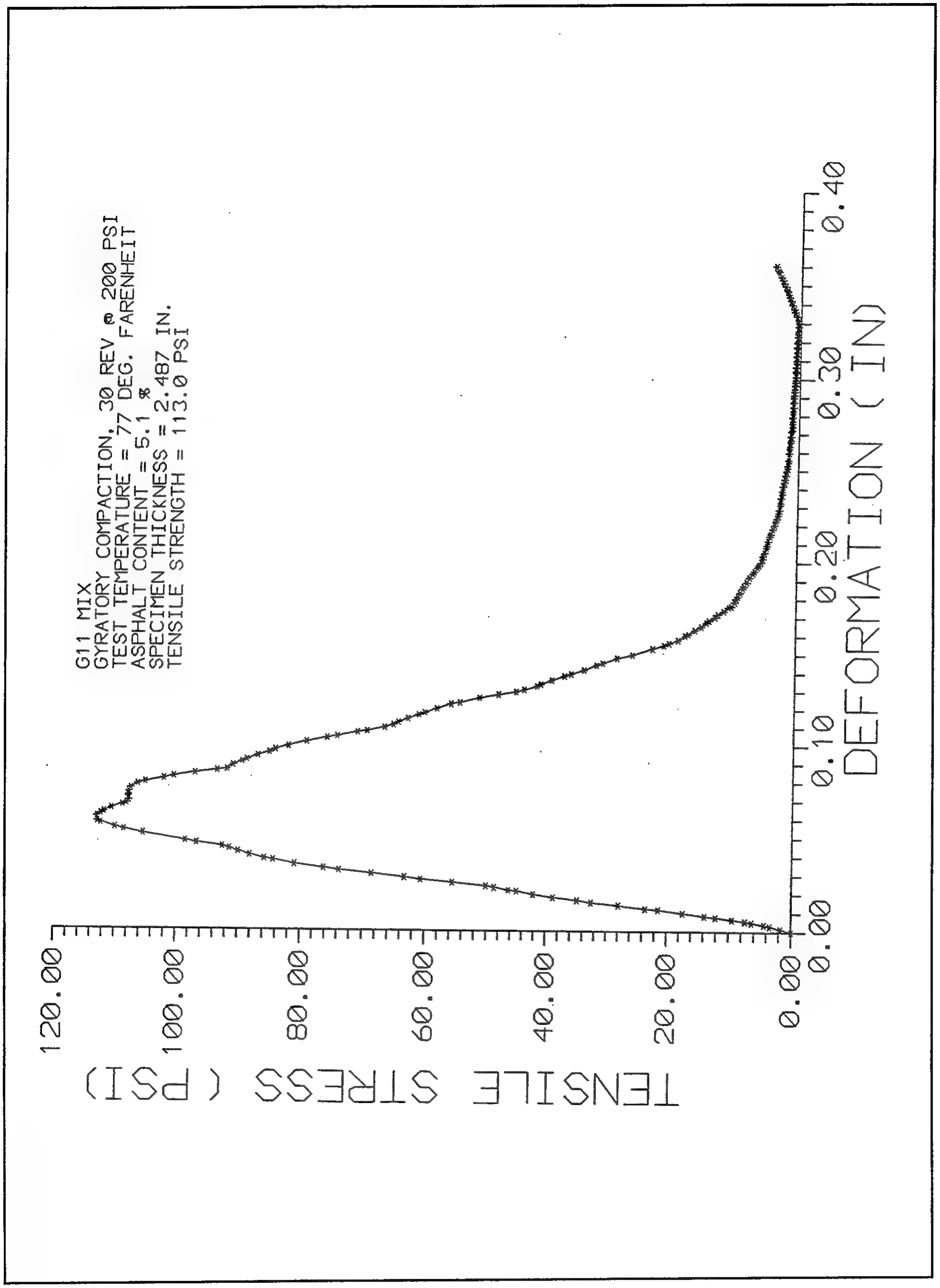

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
SPECIMEN THICKNESS = 2.499 IN.
TENSILE STRENGTH 24.8 PSI
TENSILE STRESS (PSI)
0.00
5.00
10.00
15.00
20.00
25.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

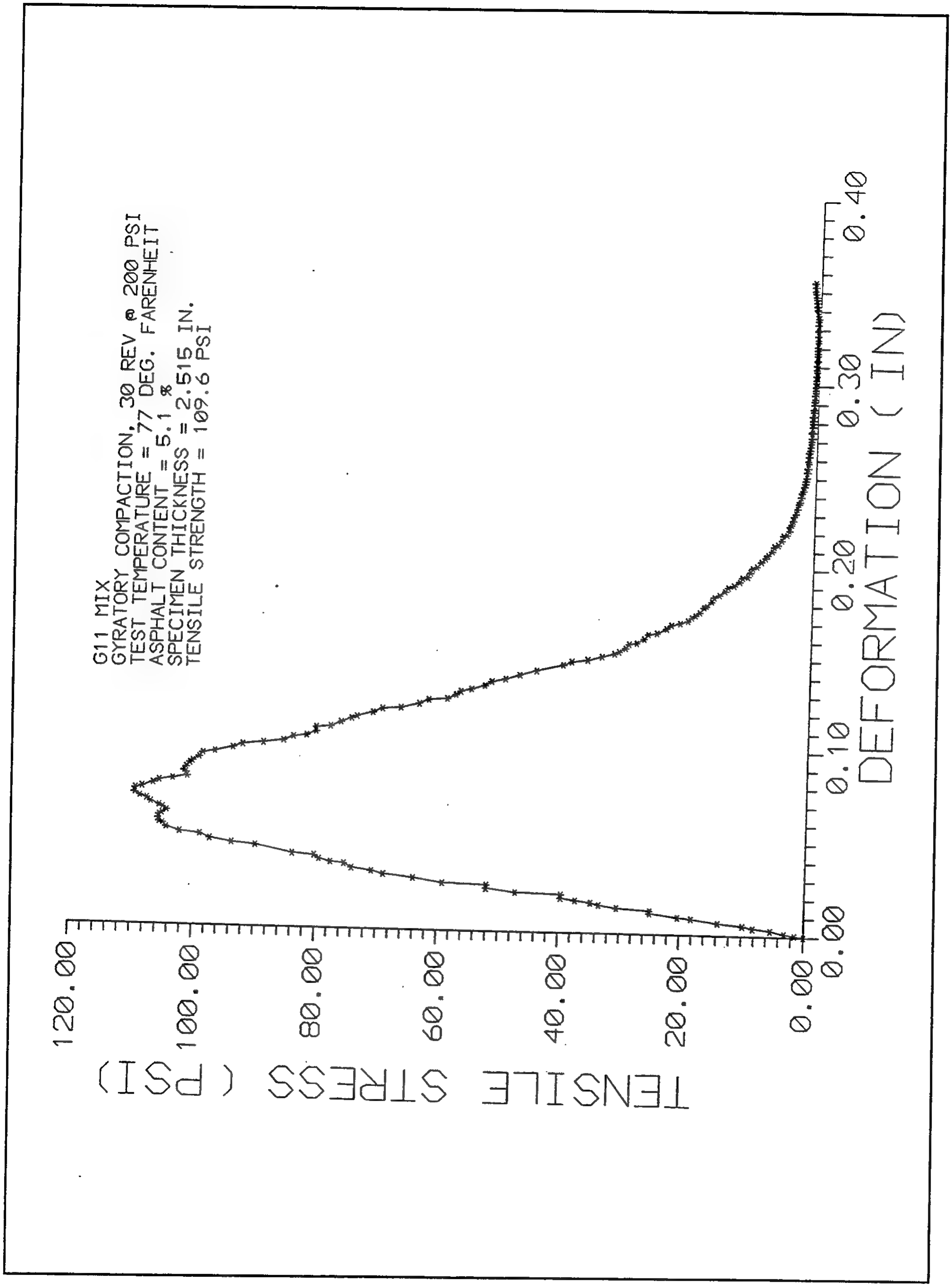

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 0 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
SPECIMEN THICKNESS = 2.541 IN.
TENSILE STRENGTH = 500.2 PSI
TENSILE STRESS (PSI)
600.00
500.00
400.00
300.00
200.00
100.00
0.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30

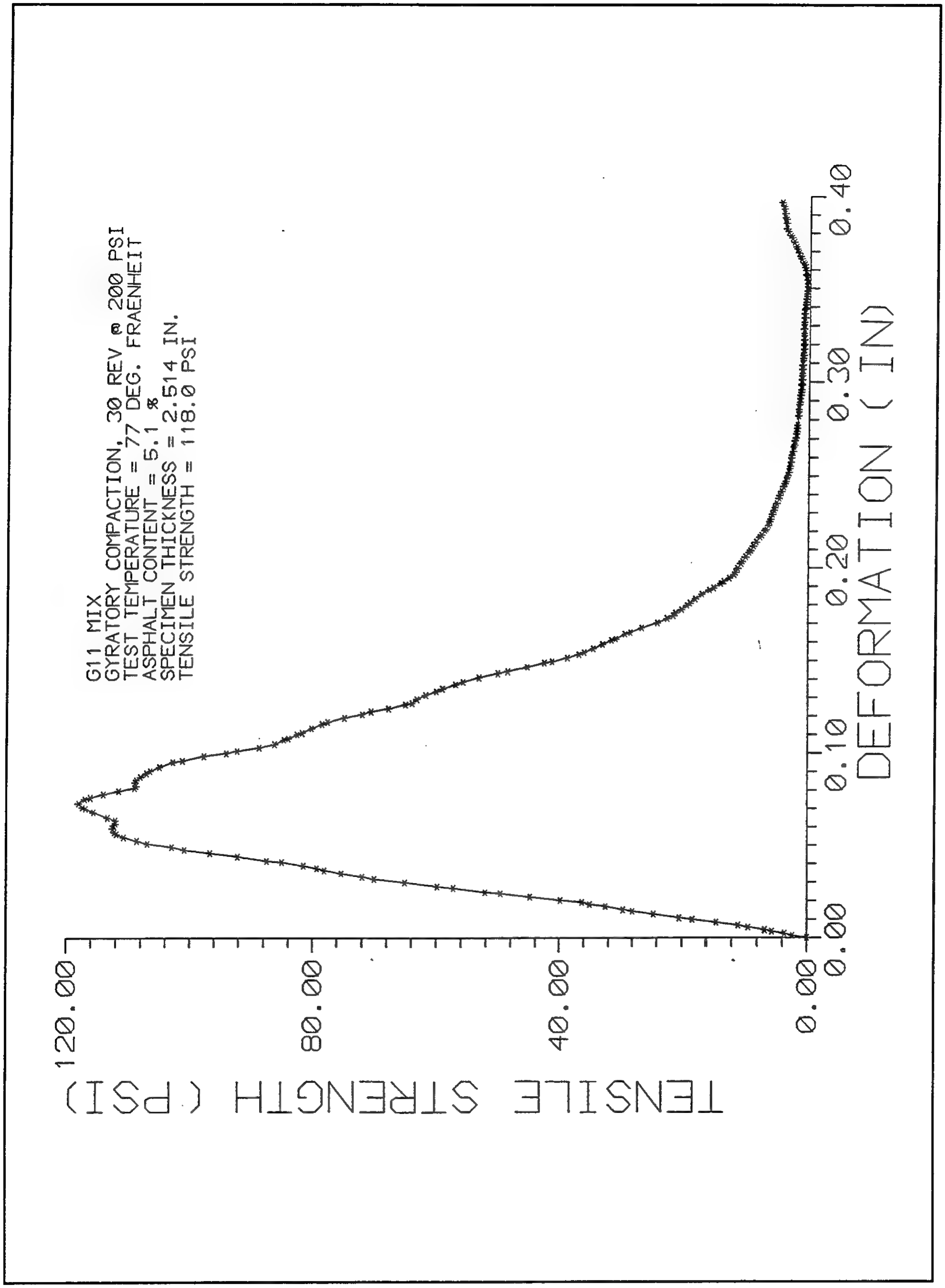

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 0 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
SPECIMEN THICKNESS = 2.503 IN.
TENSILE STRENGTH = 537.6 PSI
TENSILE STRESS (PSI)
0.00
100.00
200.00
300.00
400.00
500.00
600.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40
0.50
0.60

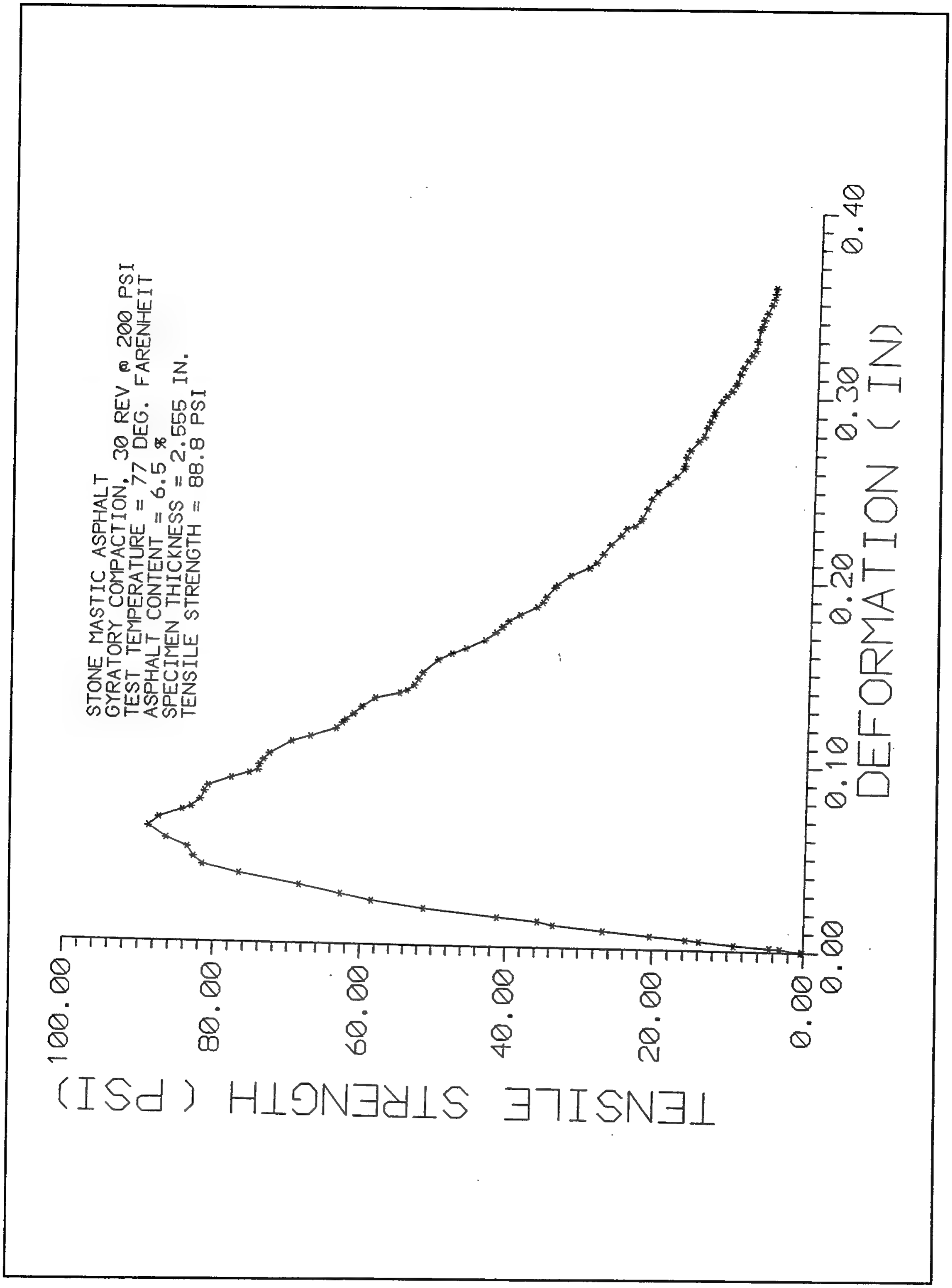

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.6 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
0.01
0.001
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)

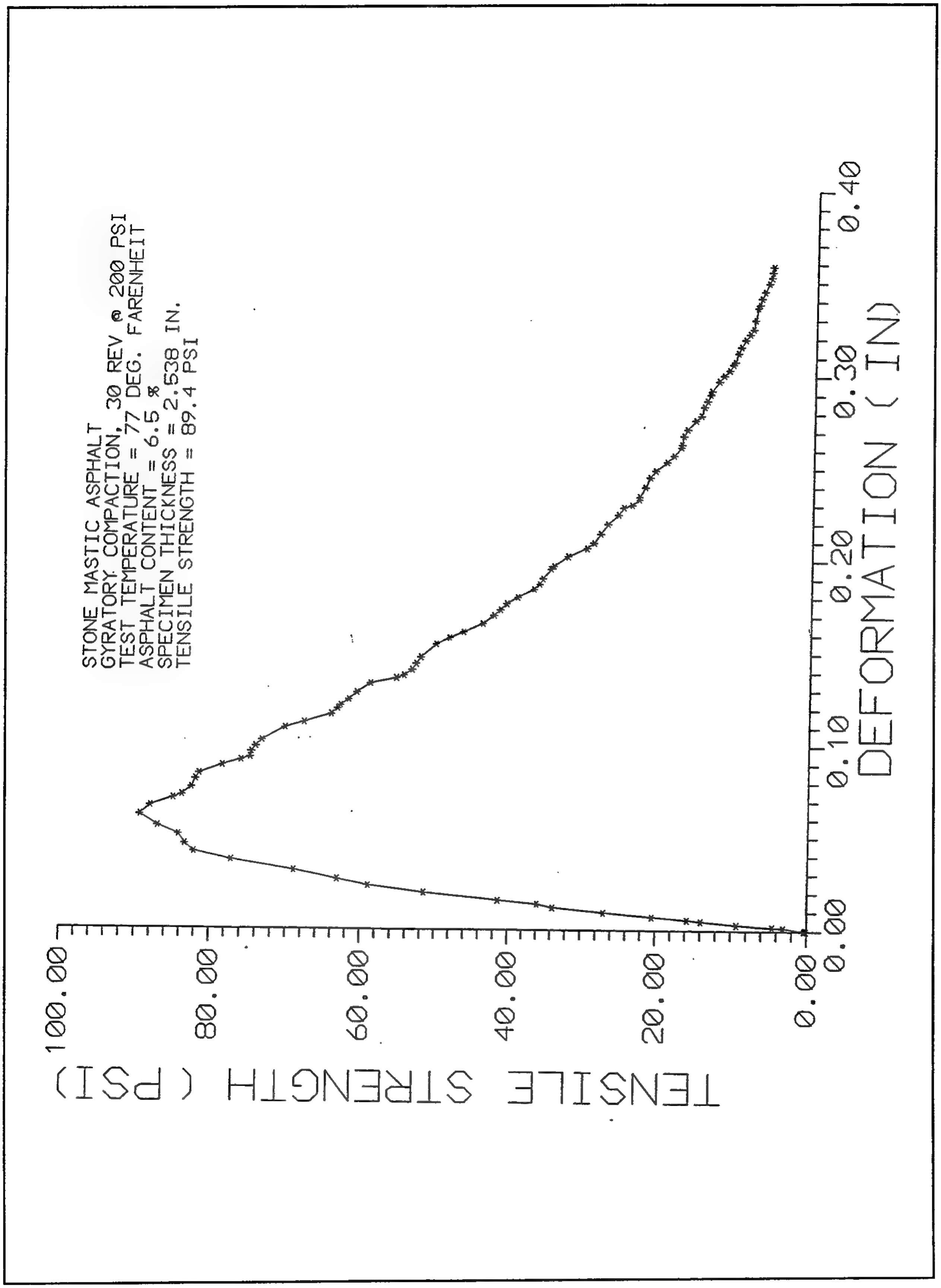

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 0 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
SPECIMEN THICKNESS = 2.445 IN.
TENSILE STRENGTH = 496.2 PSI
TENSILE STRESS (PSI)
0.00
100.00
200.00
300.00
400.00
500.00
DEFORMATION (IN)
0.00
0.05
0.10
0.15
0.20
0.25

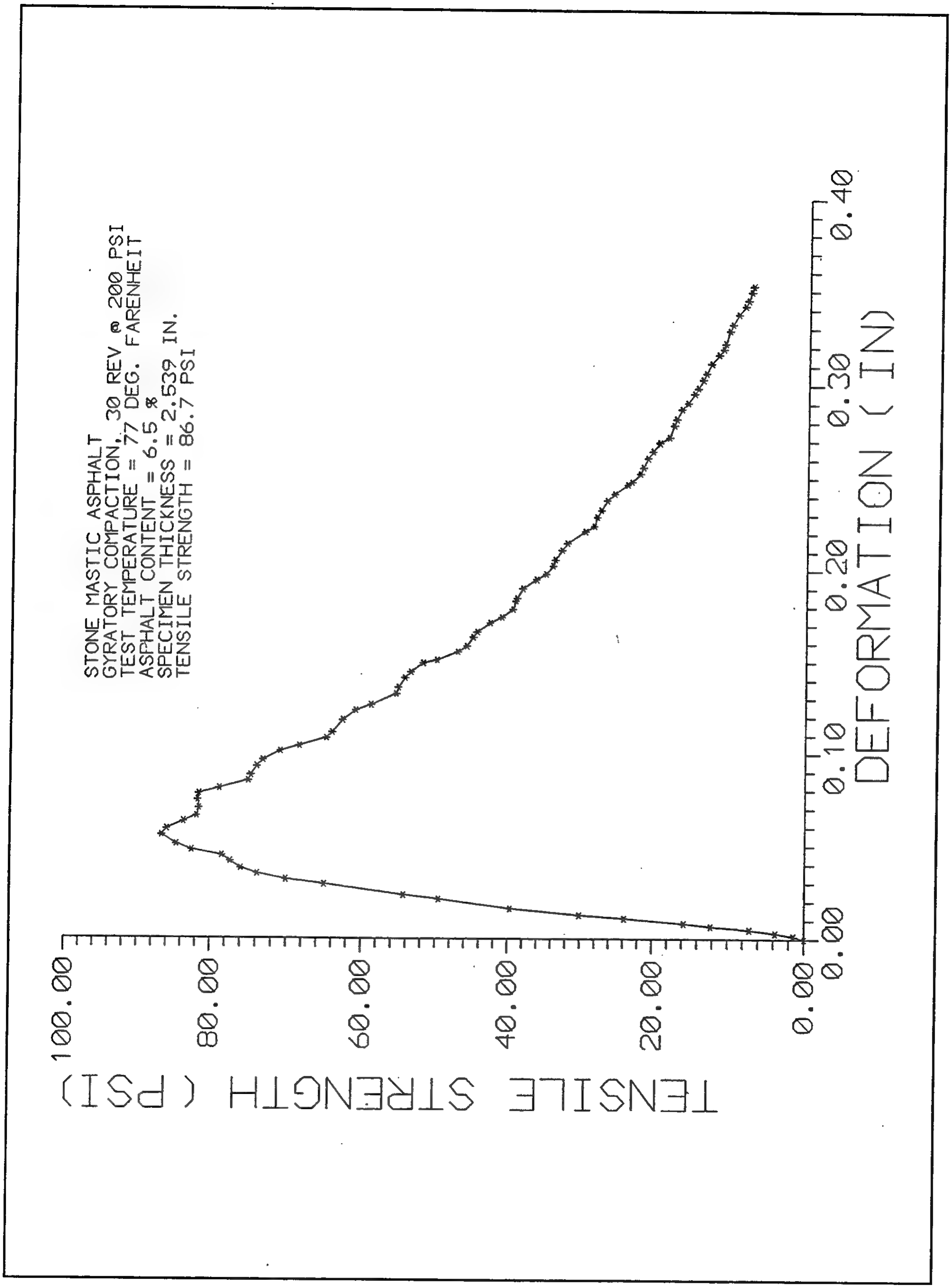

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
SPECIMEN THICKNESS = 2.487 IN.
TENSILE STRENGTH = 113.0 PSI
TENSILE STRESS (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
120.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

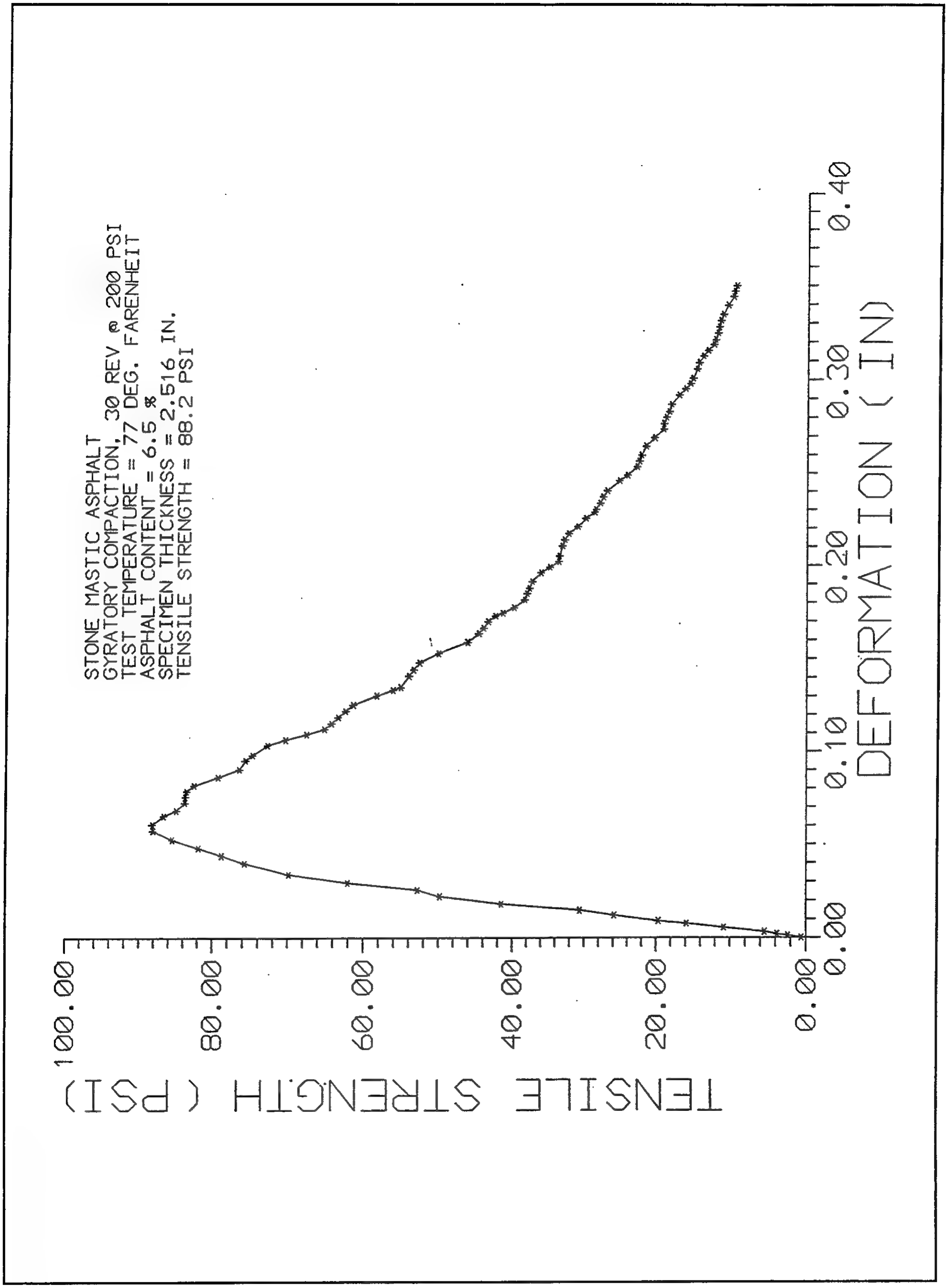

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
SPECIMEN THICKNESS = 2.515 IN.
TENSILE STRENGTH = 109.6 PSI
TENSILE STRESS (PSI)
DEFORMATION (IN)
0.00
20.00
40.00
60.00
80.00
100.00
120.00
0.00
0.10
0.20
0.30
0.40

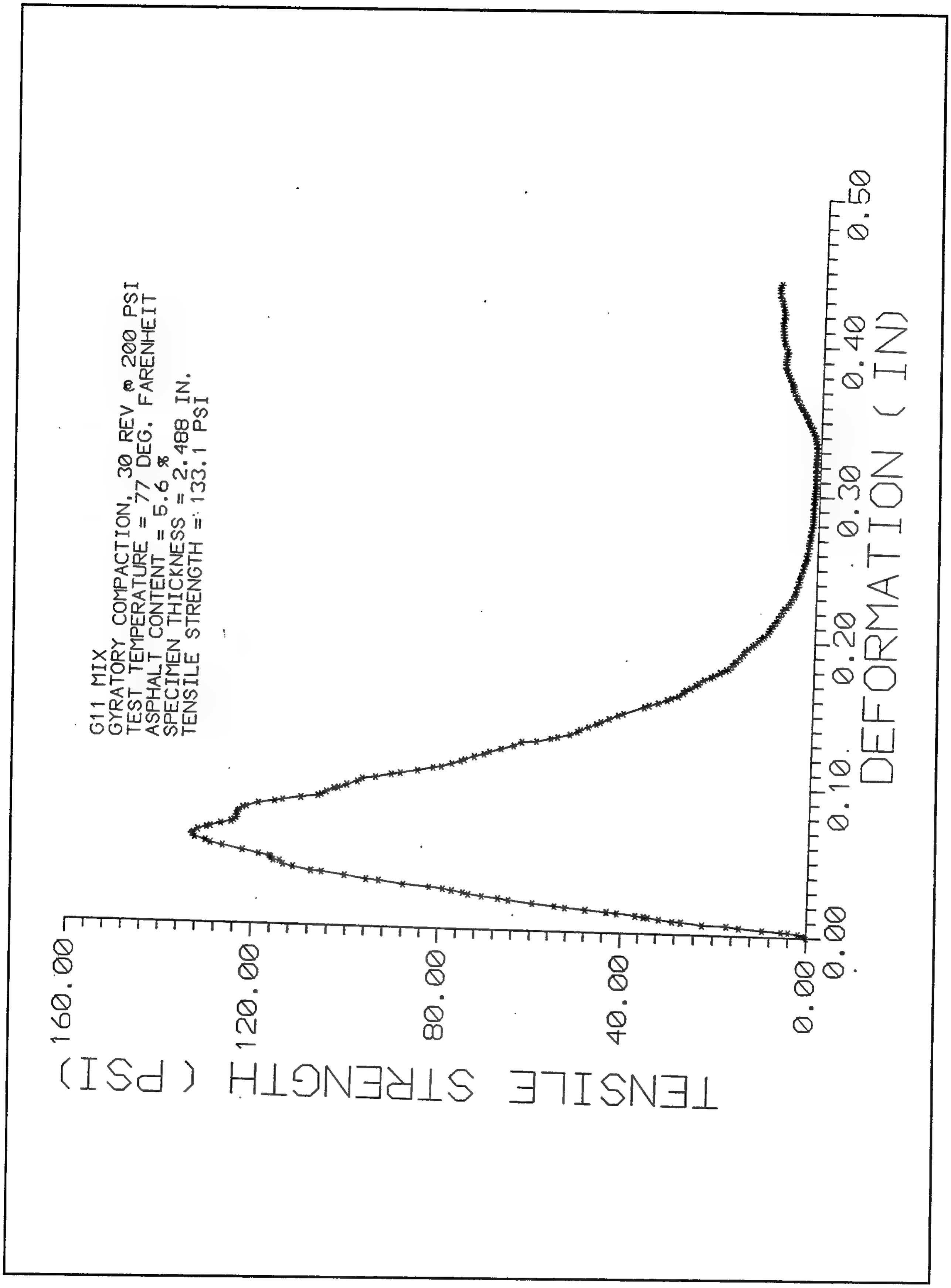

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FRAENHEIT
ASPHALT CONTENT = 5.1 %
SPECIMEN THICKNESS = 2.514 IN.
TENSILE STRENGTH = 118.0 PSI
0.00
0.10
0.20
0.30
0.40
DEFORMATION (IN)
0.00
40.00
80.00
120.00
TENSILE STRENGTH (PSI)

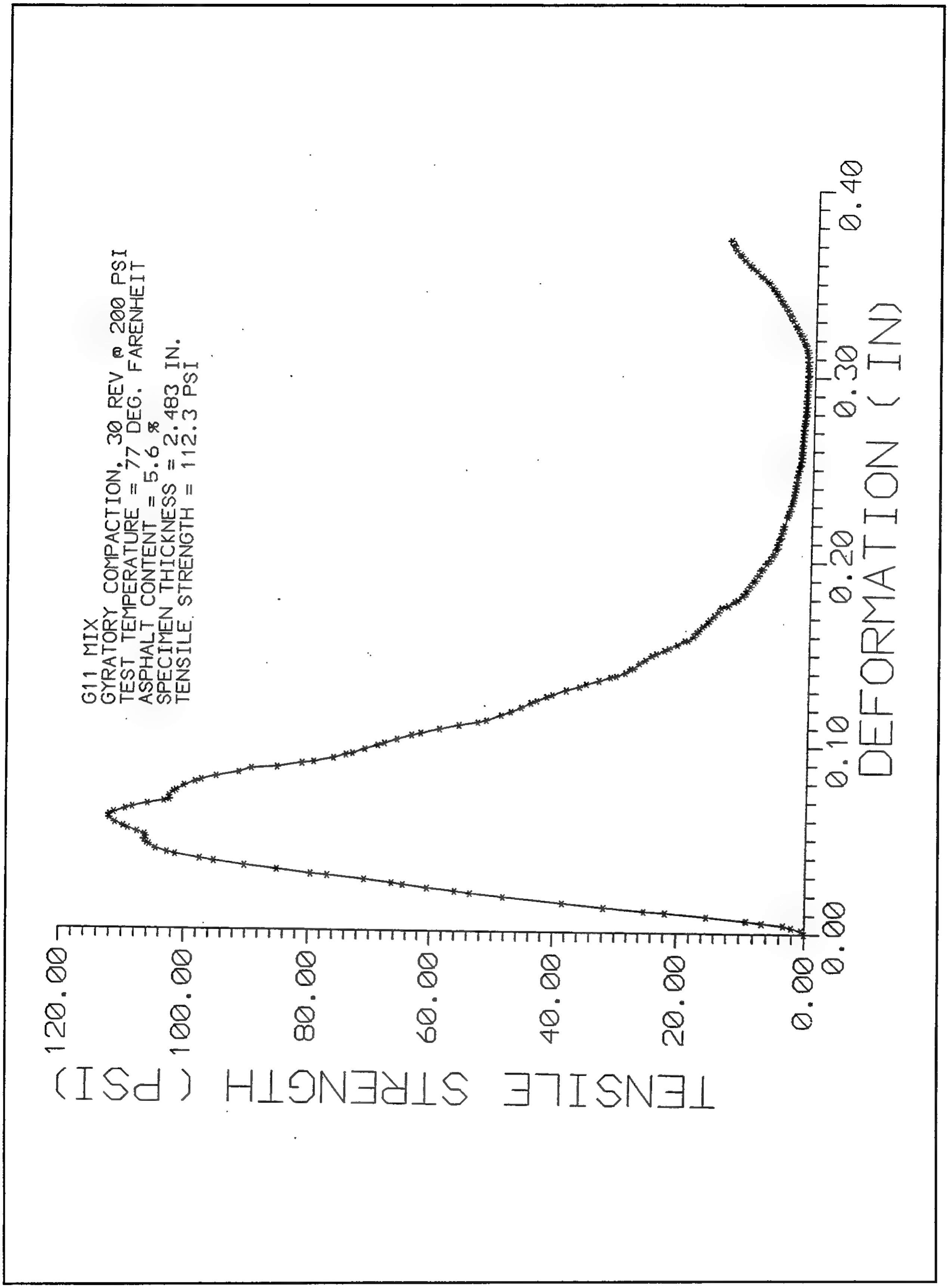

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6.5 %
SPECIMEN THICKNESS = 2.555 IN.
TENSILE STRENGTH = 88.8 PSI
TENSILE STRENGTH (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

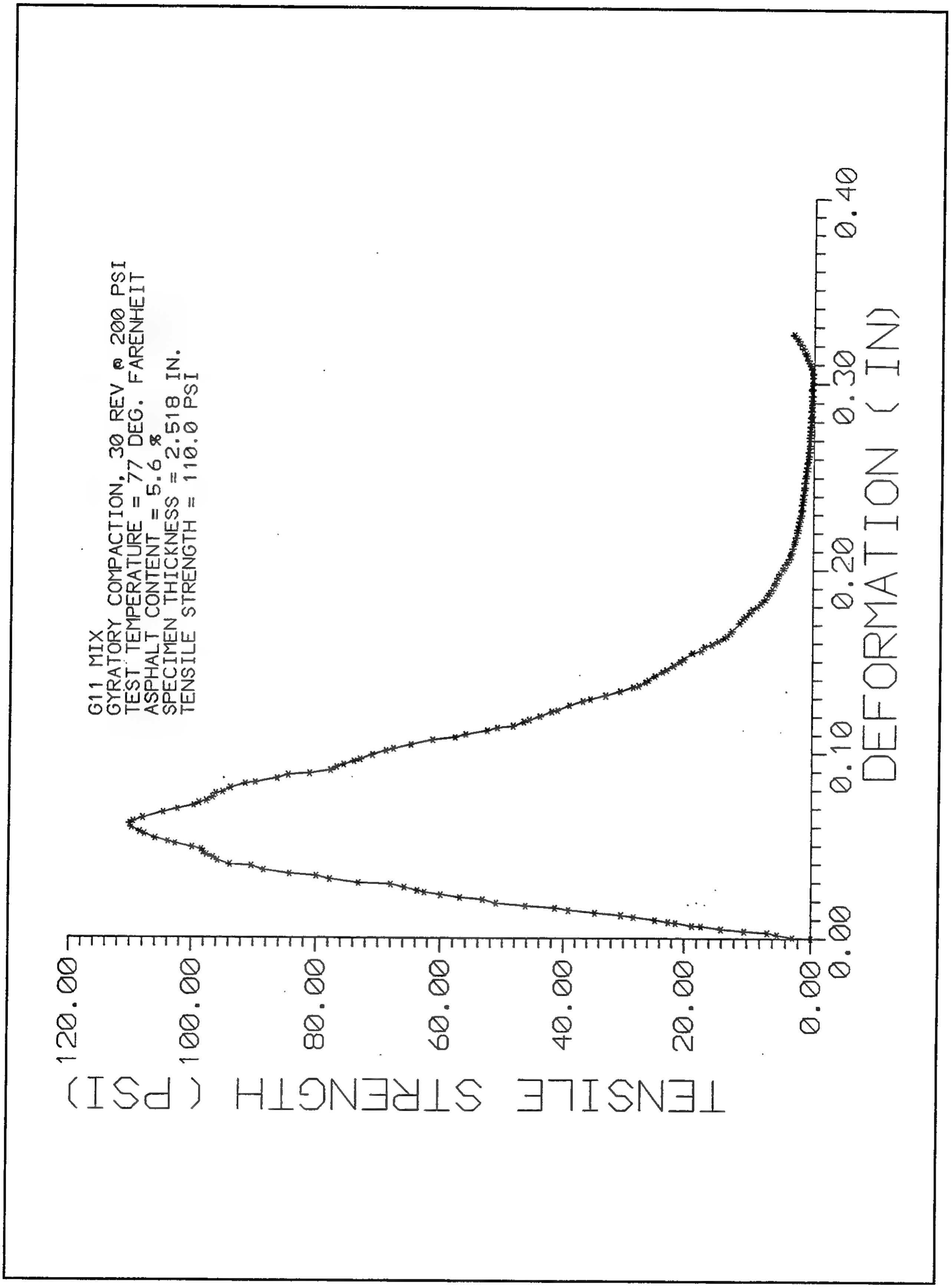

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6.5 %
SPECIMEN THICKNESS = 2.538 IN.
TENSILE STRENGTH = 89.4 PSI
TENSILE STRENGTH (PSI)
100.00
80.00
60.00
40.00
20.00
0.00
0.00
0.10
0.20
0.30
0.40
DEFORMATION (IN)


STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6.5 %
SPECIMEN THICKNESS = 2.539 IN.
TENSILE STRENGTH = 86.7 PSI
TENSILE STRENGTH (PSI)
100.00
80.00
60.00
40.00
20.00
0.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

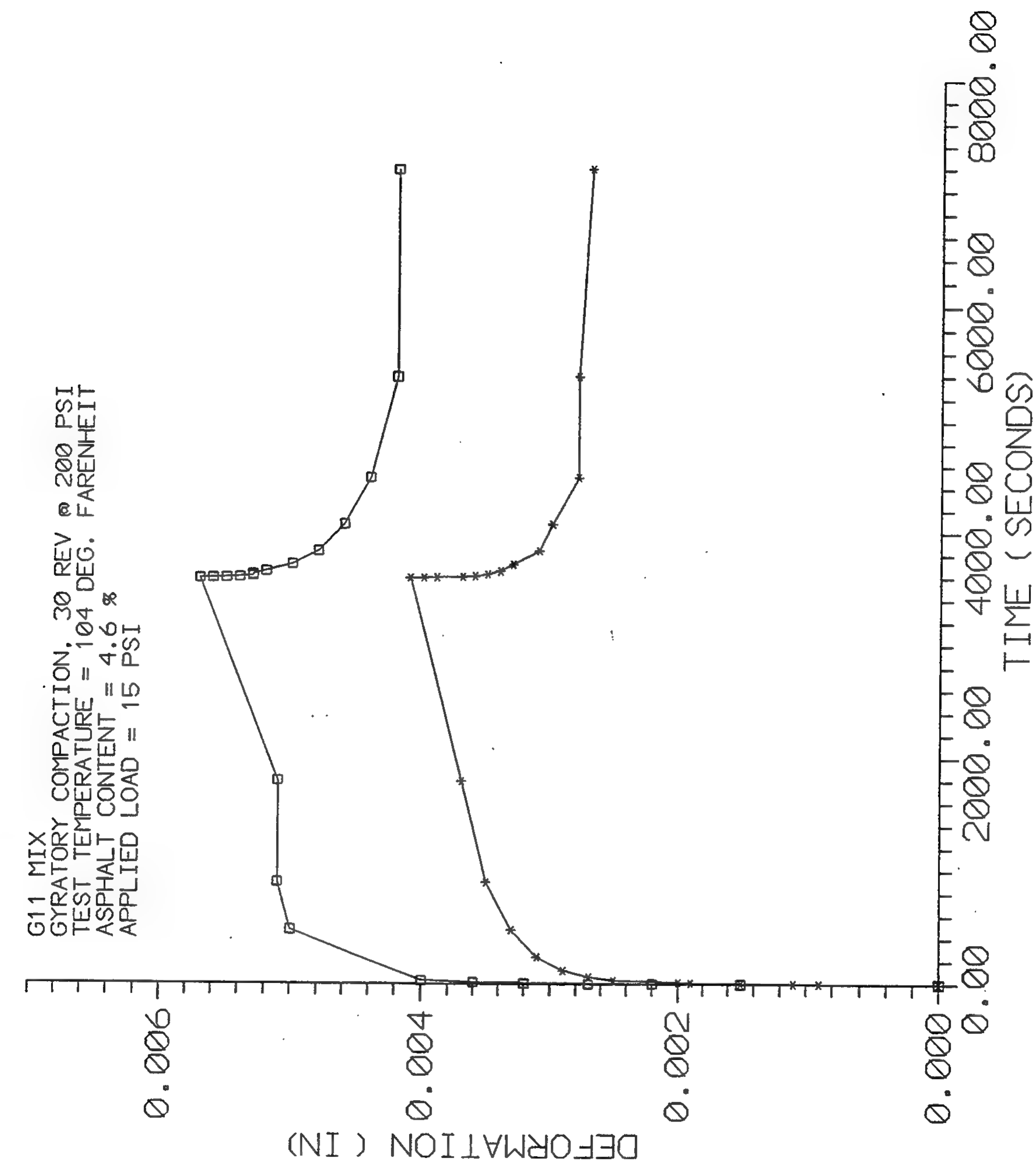

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6.5 %
SPECIMEN THICKNESS = 2.516 IN.
TENSILE STRENGTH = 88.2 PSI
TENSILE STRENGTH (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

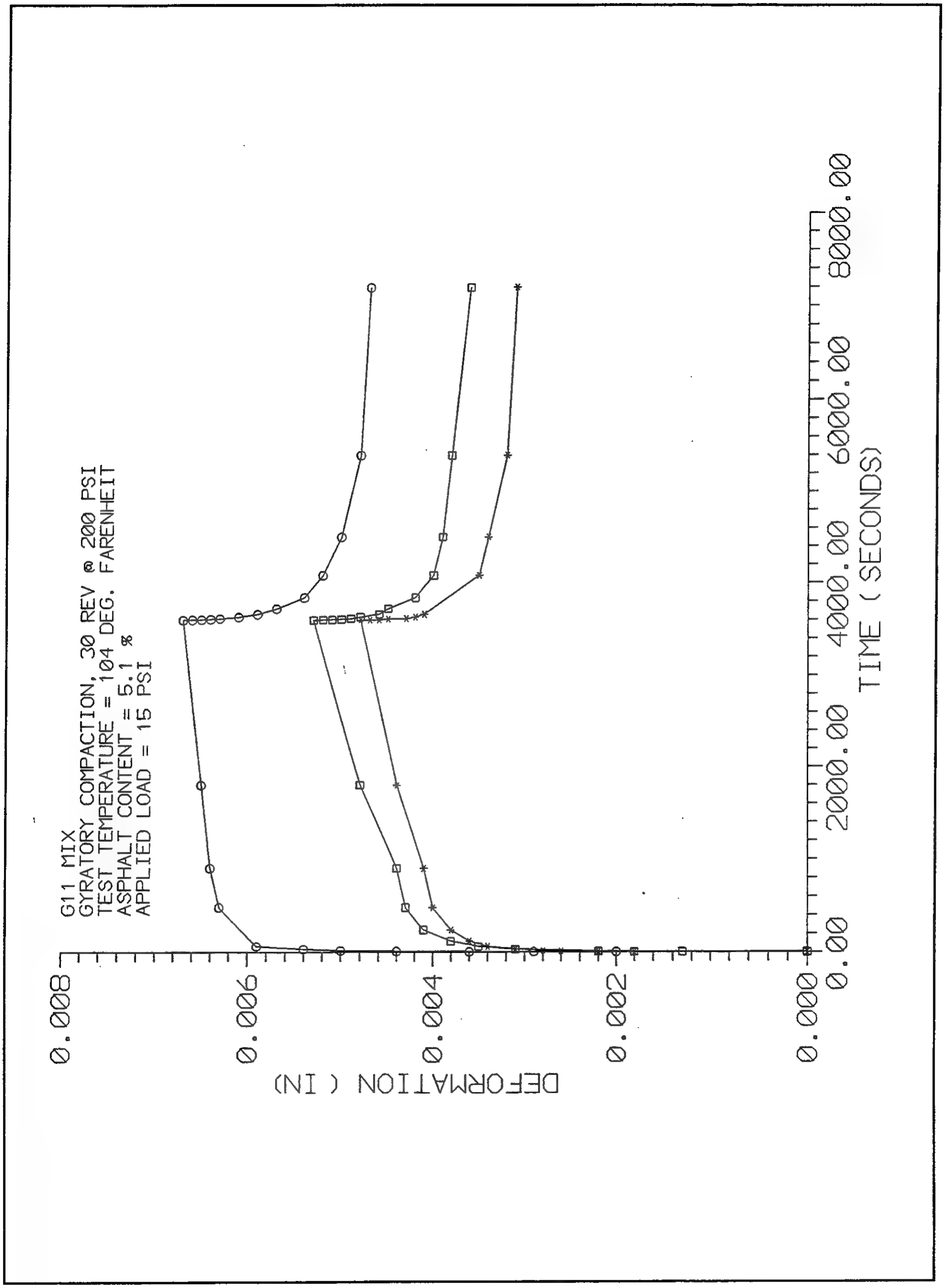

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.6 %
SPECIMEN THICKNESS = 2.488 IN.
TENSILE STRENGTH = 133.1 PSI
TENSILE STRENGTH (PSI)
0.00
40.00
80.00
120.00
160.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40
0.50

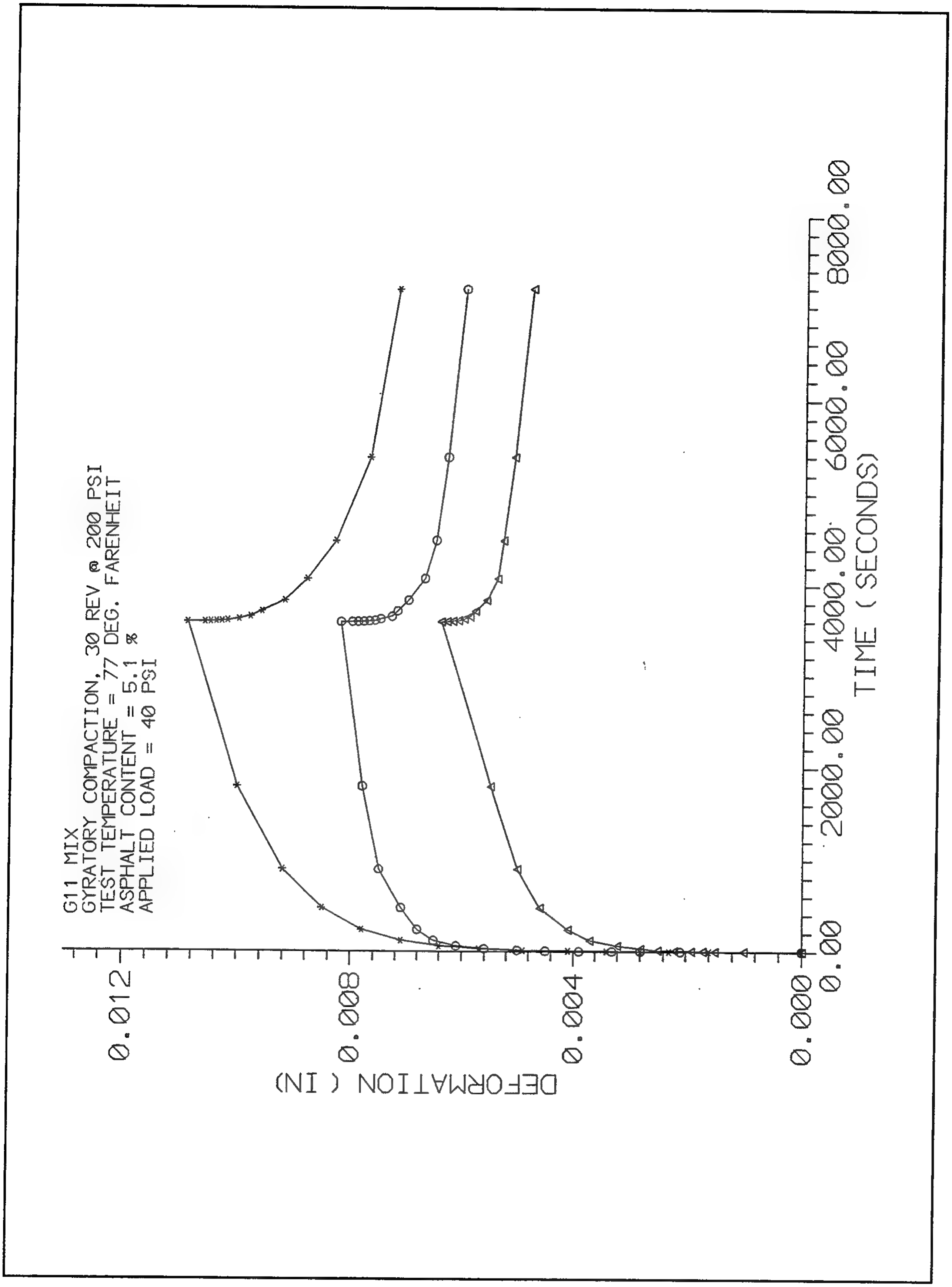

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.6 %
SPECIMEN THICKNESS = 2.483 IN.
TENSILE STRENGTH = 112.3 PSI
TENSILE STRENGTH (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
120.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

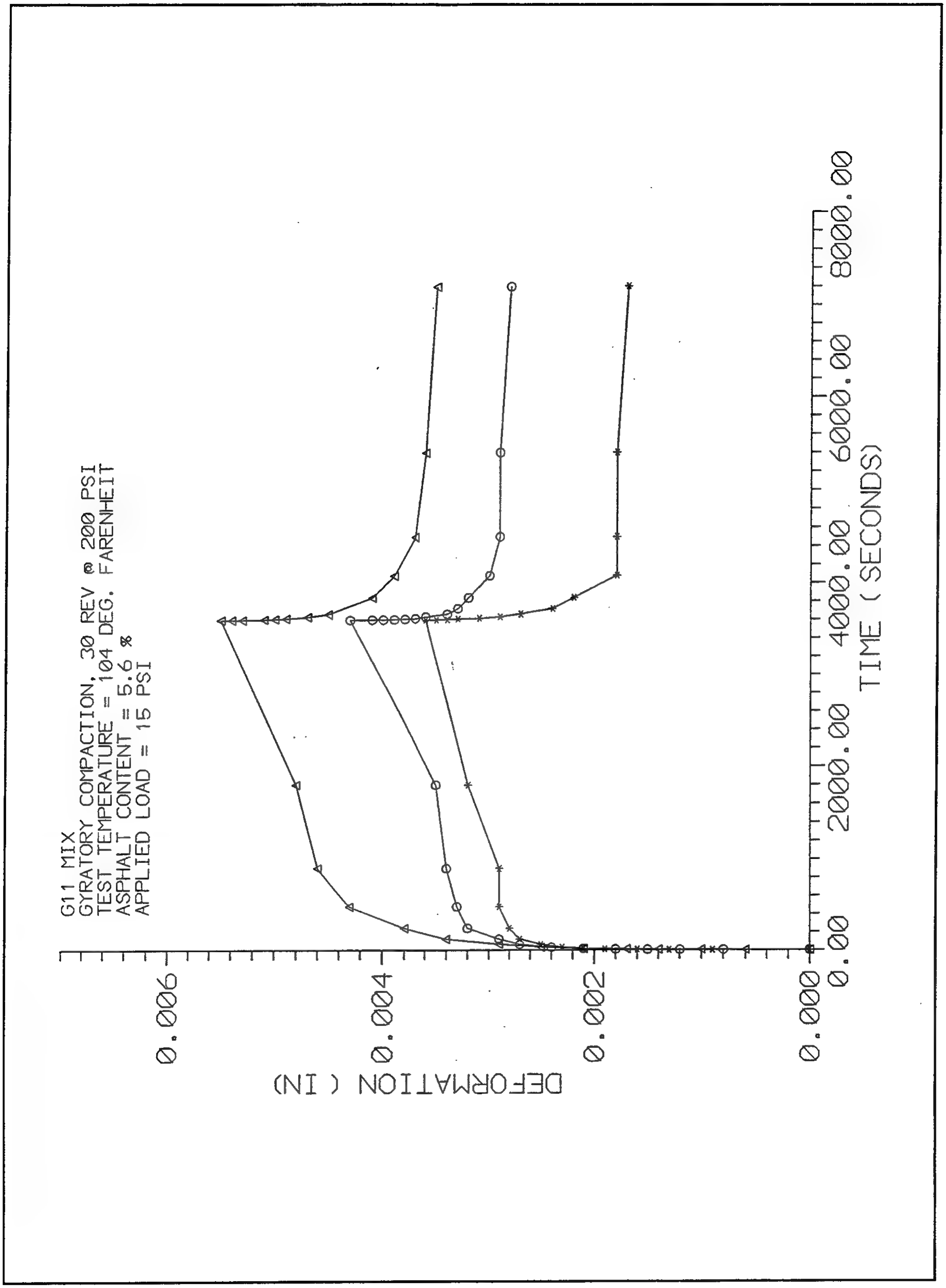

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.6 %
SPECIMEN THICKNESS = 2.518 IN.
TENSILE STRENGTH = 110.0 PSI
TENSILE STRENGTH (PSI)
0.00
20.00
40.00
60.00
80.00
100.00
120.00
DEFORMATION (IN)
0.00
0.10
0.20
0.30
0.40

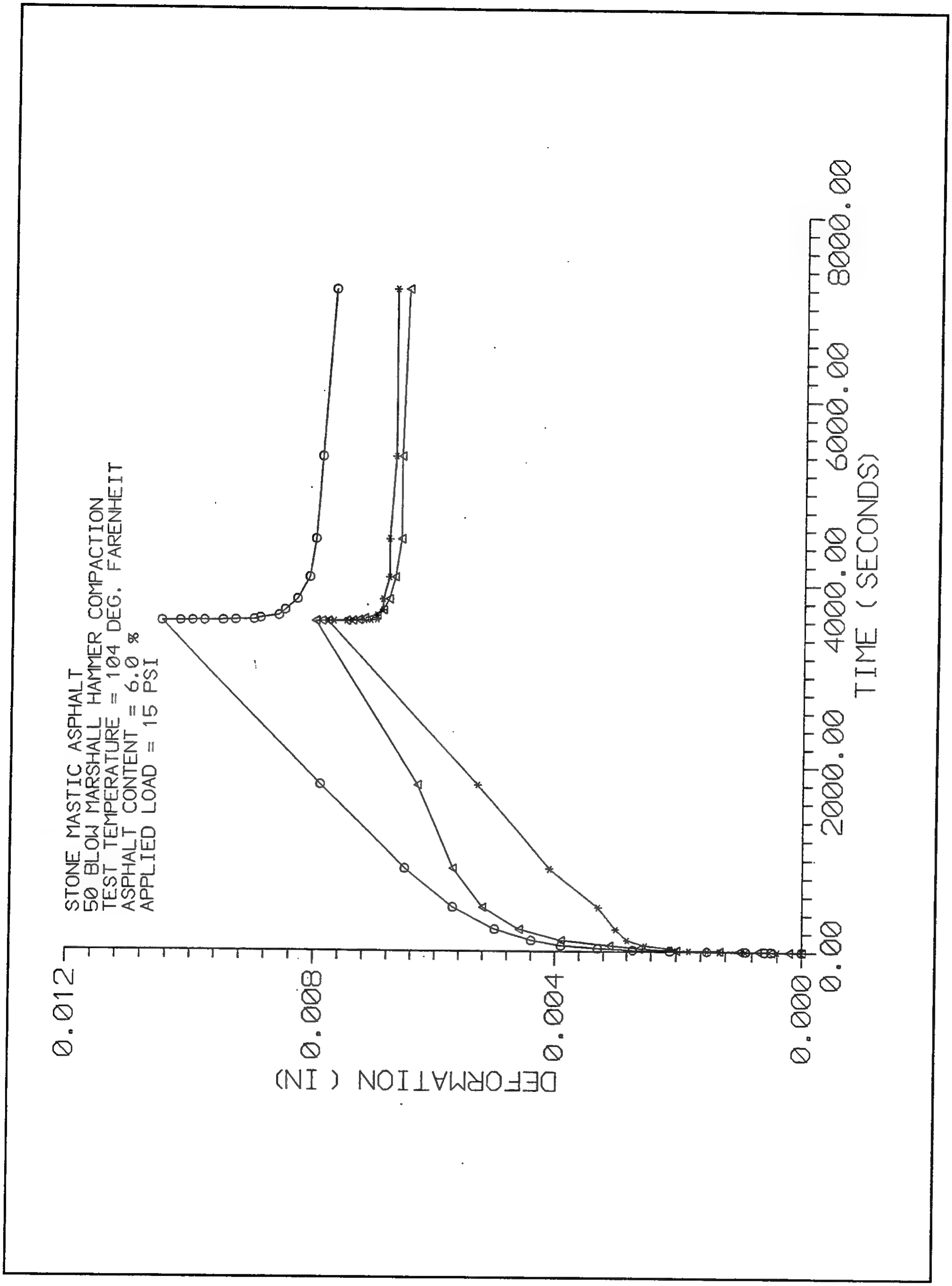
Appendix B
Unconfined Creep-Rebound Tests

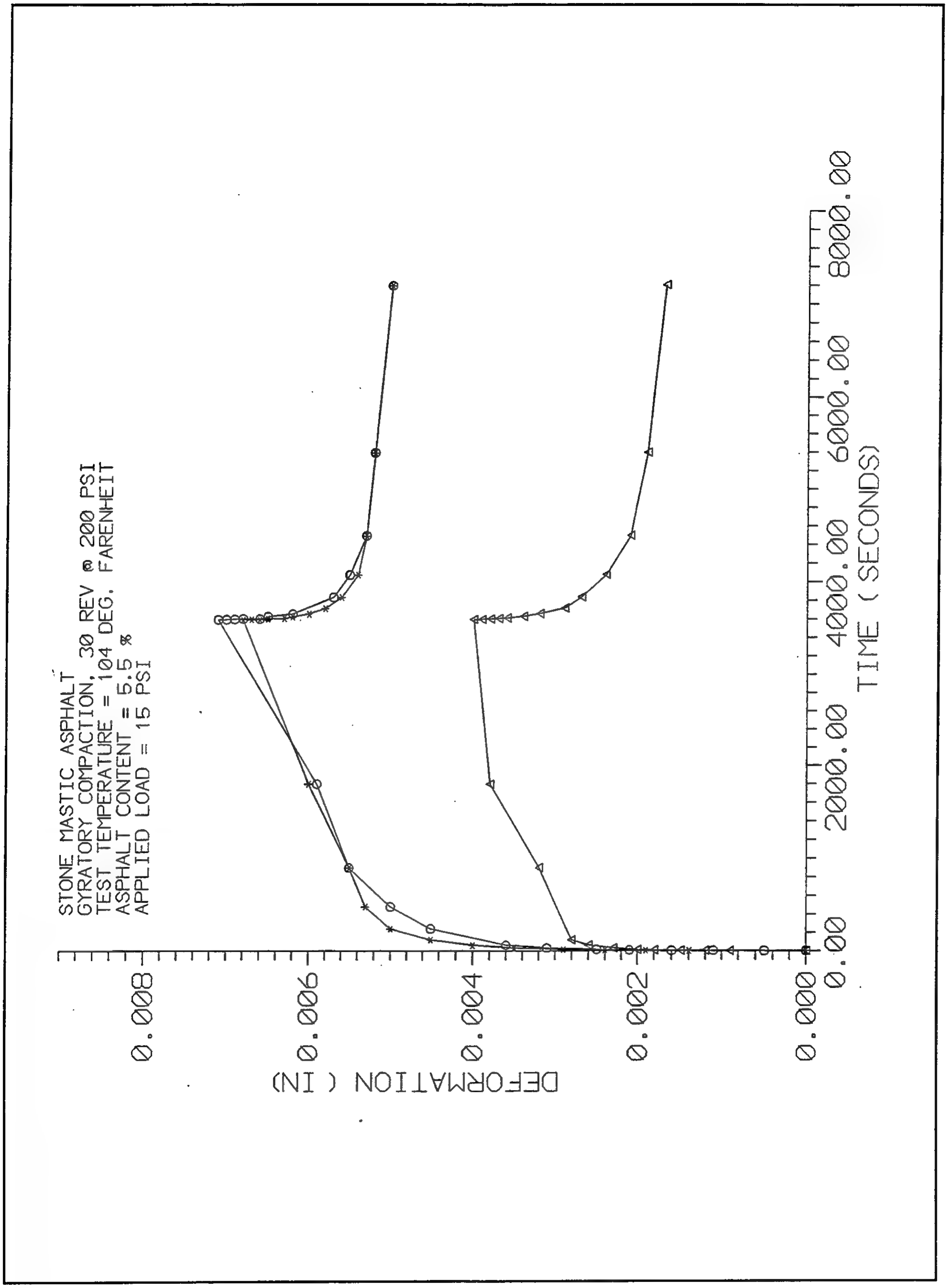

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 4.6 %
APPLIED LOAD = 15 PSI
DEFORMATION (IN)
0.000
0.002
0.004
0.006
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00

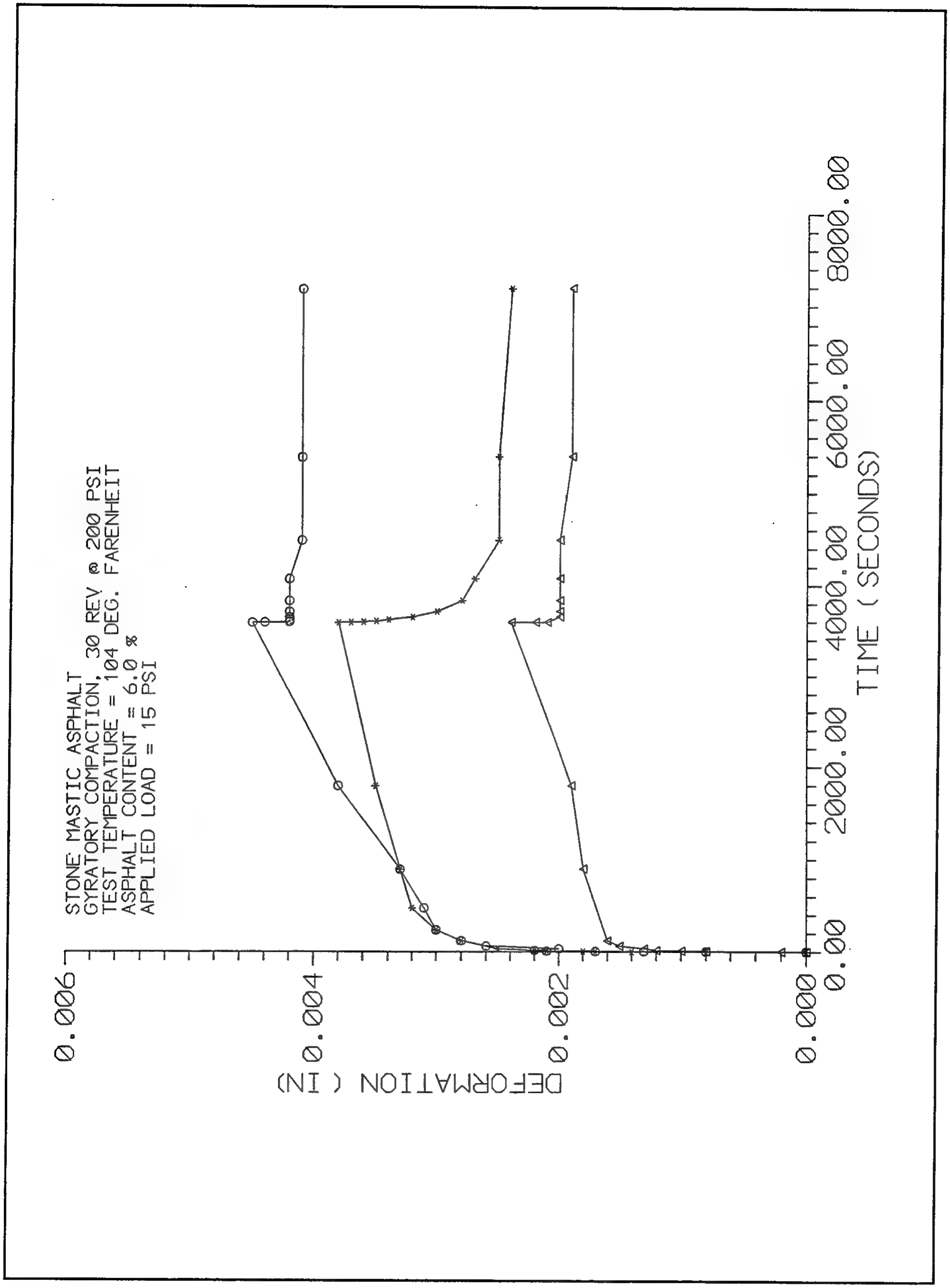

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
APPLIED LOAD = 15 PSI
DEFORMATION (IN)
0.000
0.002
0.004
0.006
0.008
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00

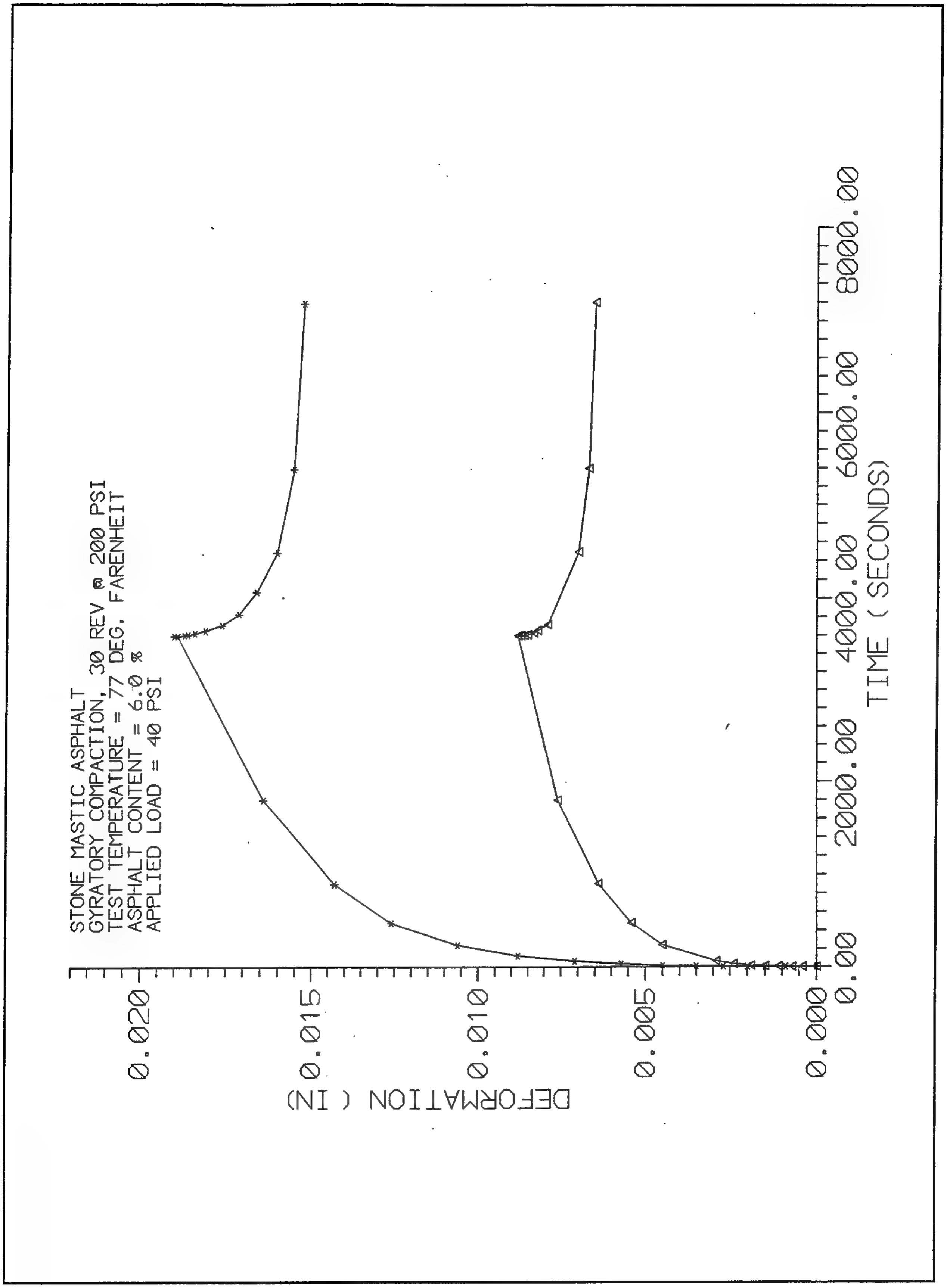

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
APPLIED LOAD = 40 PSI
DEFORMATION (IN)
0.000
0.004
0.008
0.012
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00

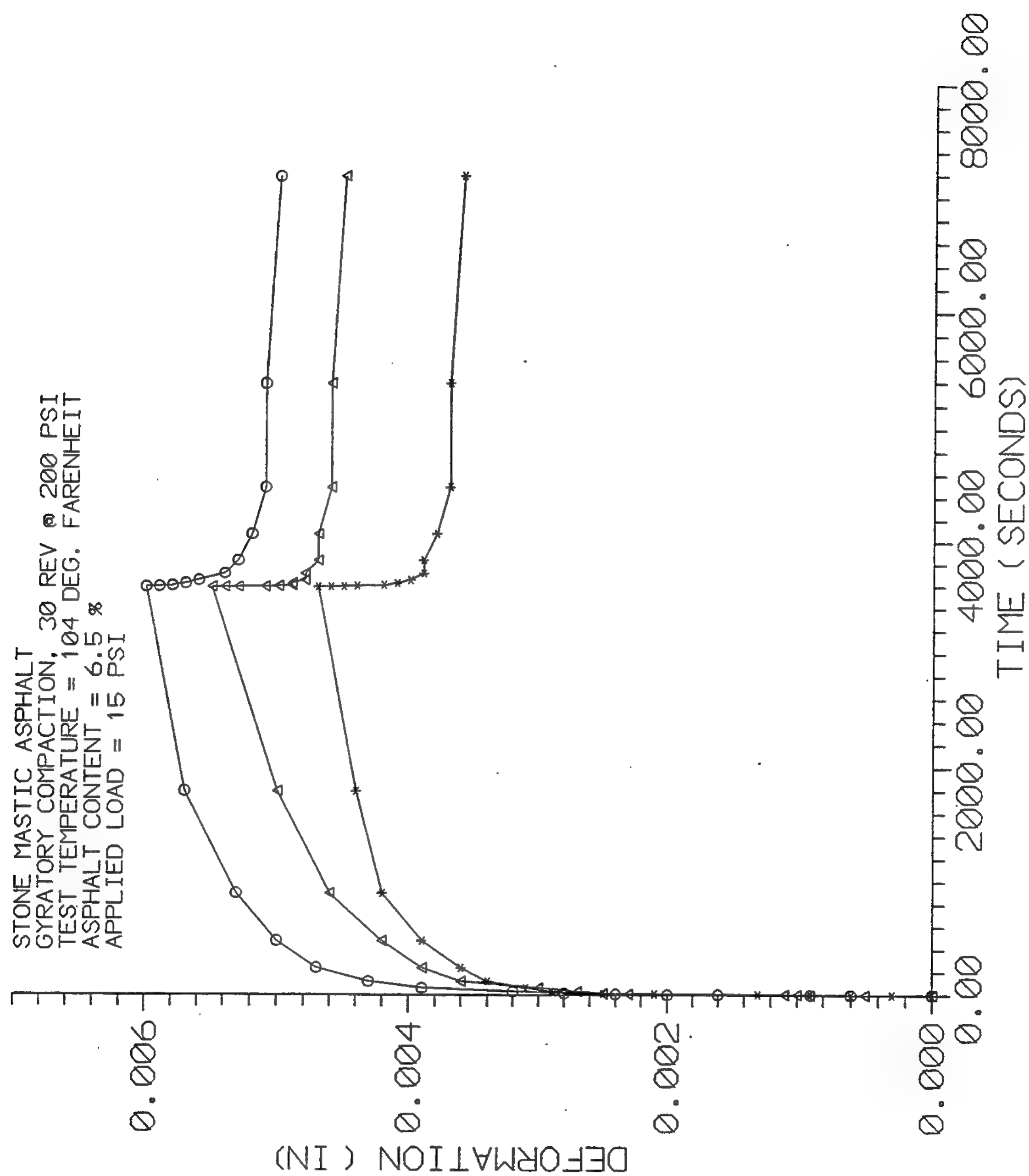

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.6 %
APPLIED LOAD = 15 PSI
DEFORMATION (IN)
0.000
0.002
0.004
0.006
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00


STONE MASTIC ASPHALT
50 BLOW MARSHALL HAMMER COMPACTION
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
APPLIED LOAD = 15 PSI
DEFORMATION (IN)
0.000
0.004
0.008
0.012
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00

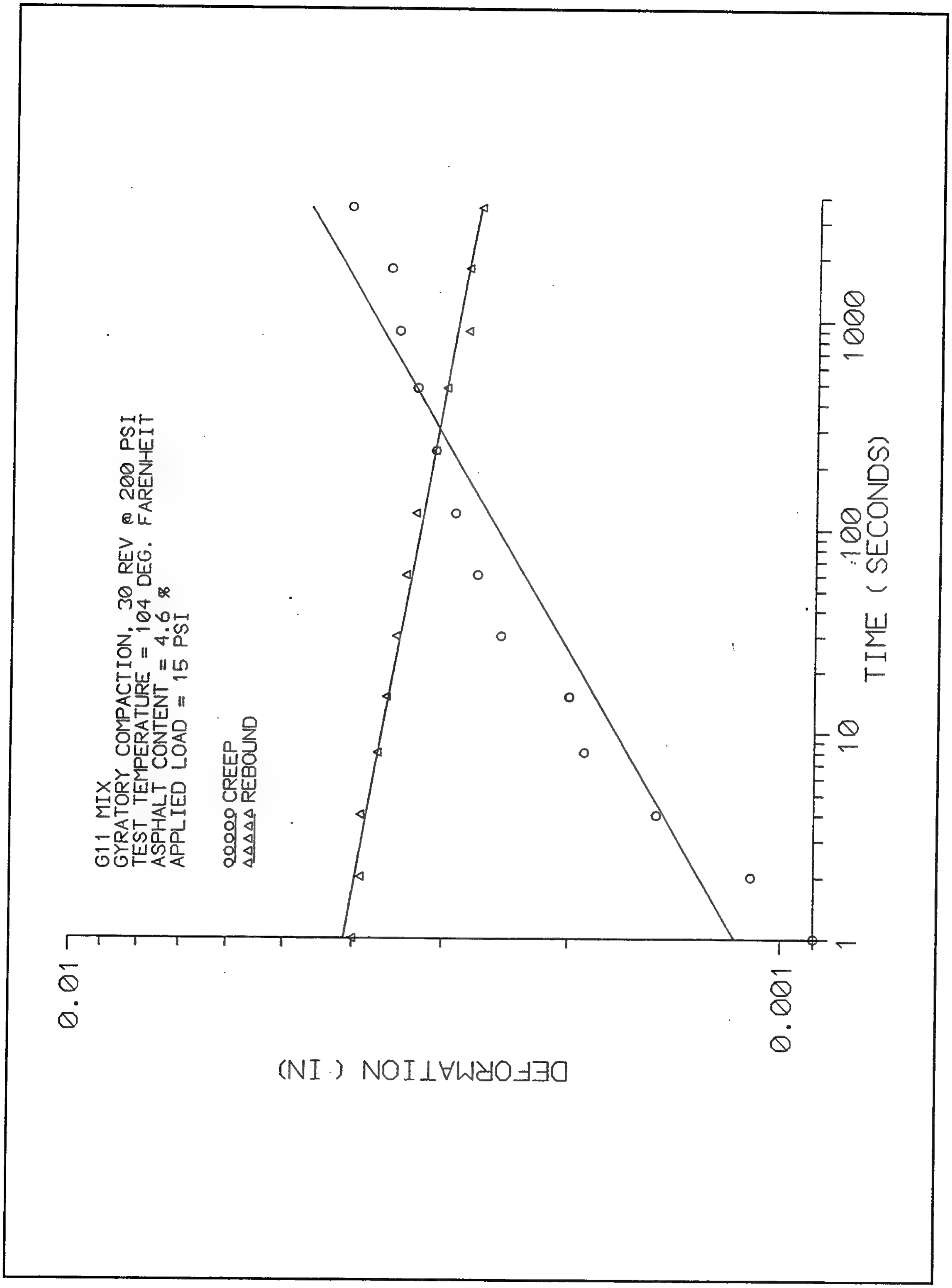

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.5 %
APPLIED LOAD = 15 PSI
DEFORMATION (IN)
0.000
0.002
0.004
0.006
0.008
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00

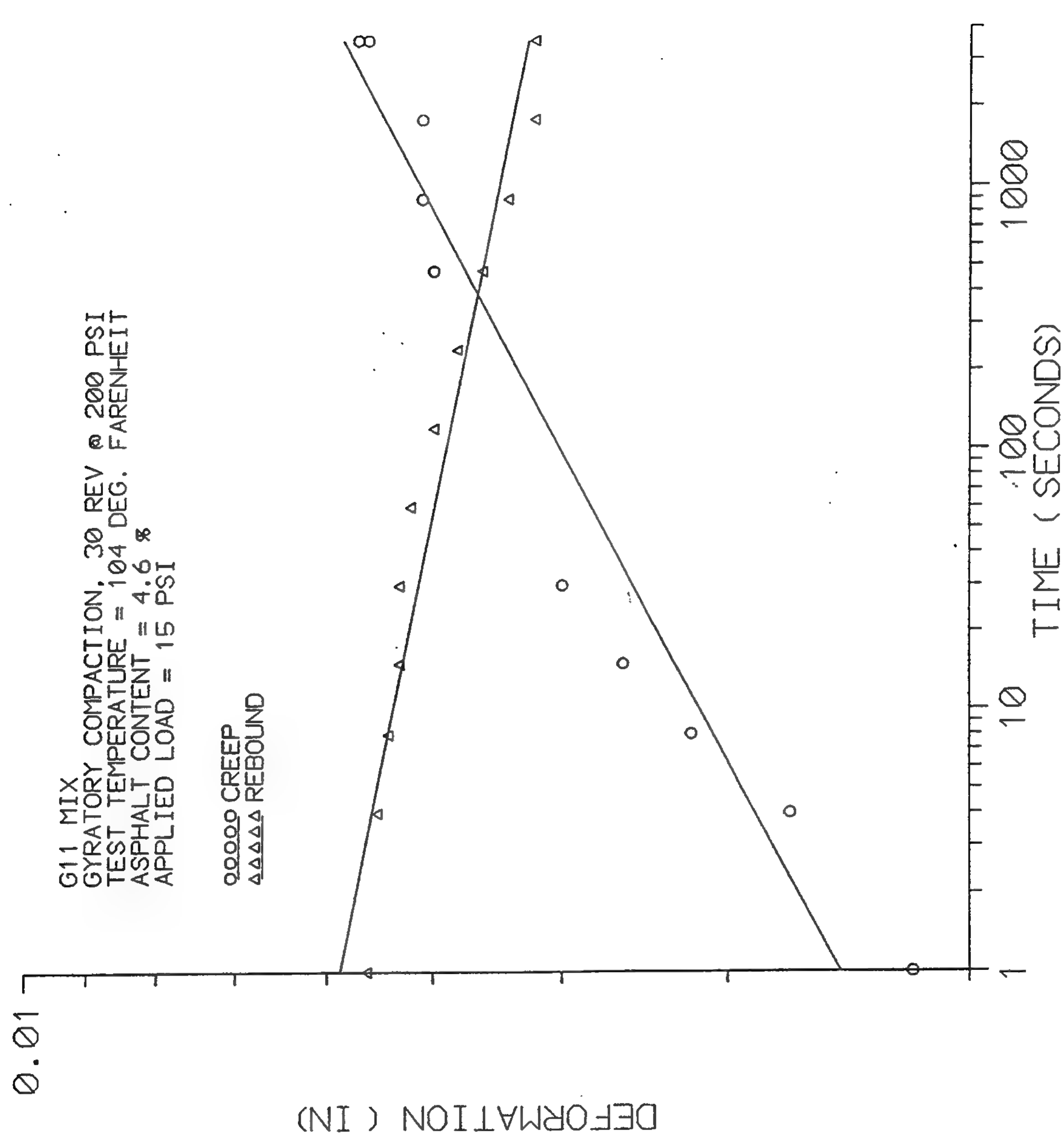

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
APPLIED LOAD = 15 PSI
DEFORMATION (IN)
0.000
0.002
0.004
0.006
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00

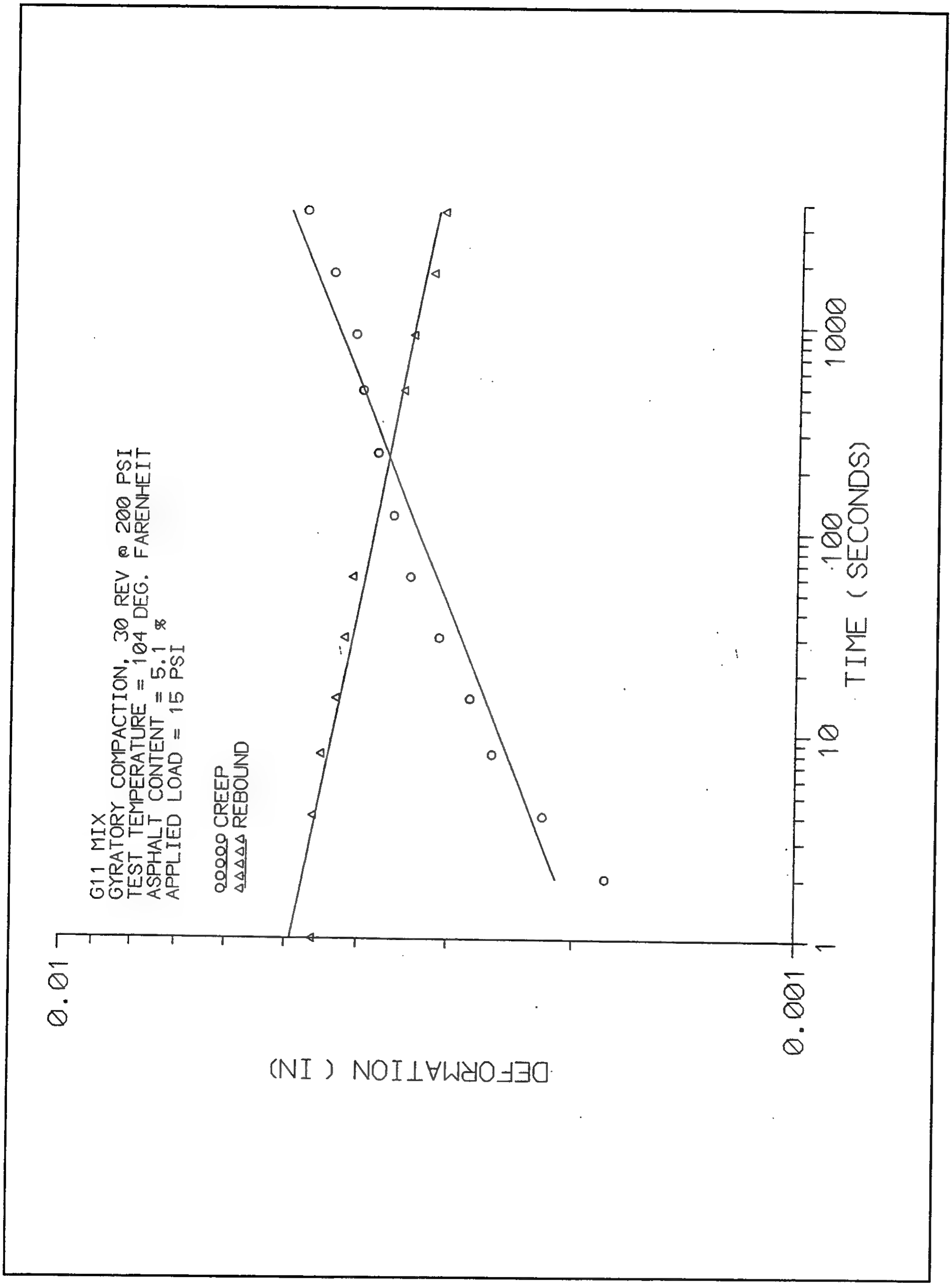

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
APPLIED LOAD = 40 PSI
DEFORMATION (IN)
0.000
0.005
0.010
0.015
0.020
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00

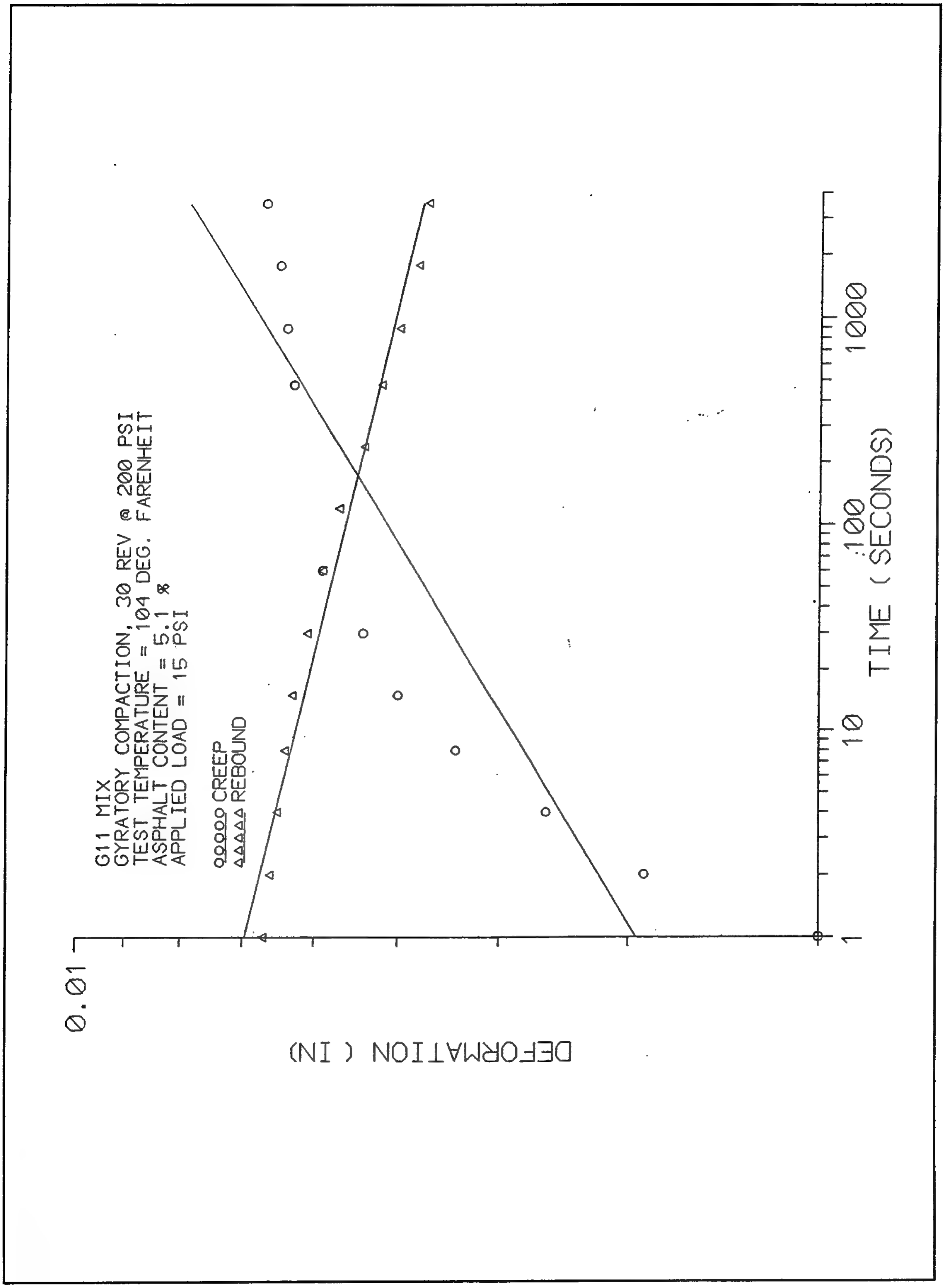

STONE MASTIC ASPHALT
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.5 %
APPLIED LOAD = 15 PSI
DEFORMATION (IN)
0.000
0.002
0.004
0.006
TIME (SECONDS)
0.00
2000.00
4000.00
6000.00
8000.00

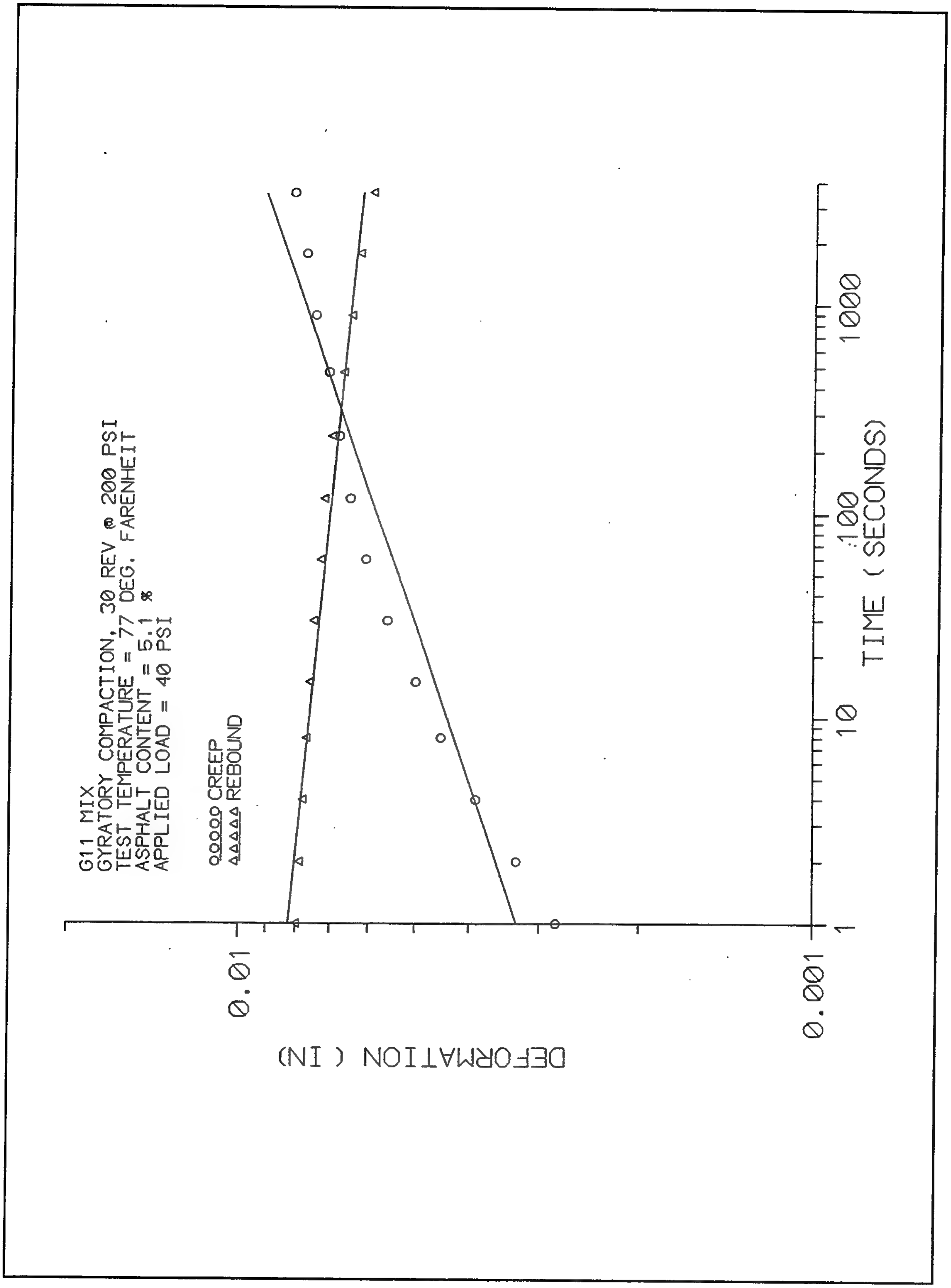
Appendix C
Power Curvefit of Creep-Rebound Tests

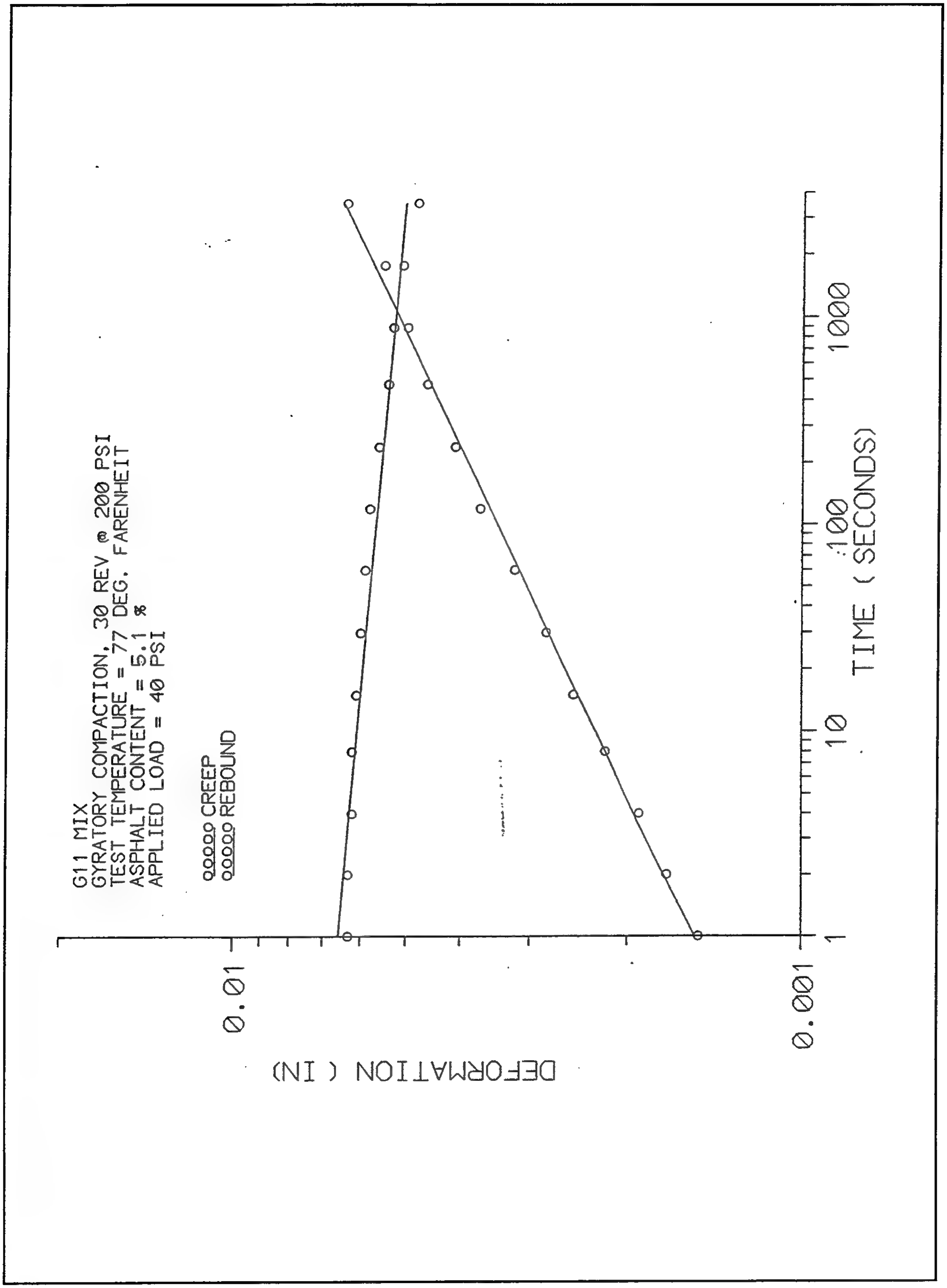

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 4.6 %
APPLIED LOAD = 15 PSI
OOOOO CREEP
ΔΔΔΔΔ REBOUND
0.01
0.001
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)


G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 4.6 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
0.01
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)

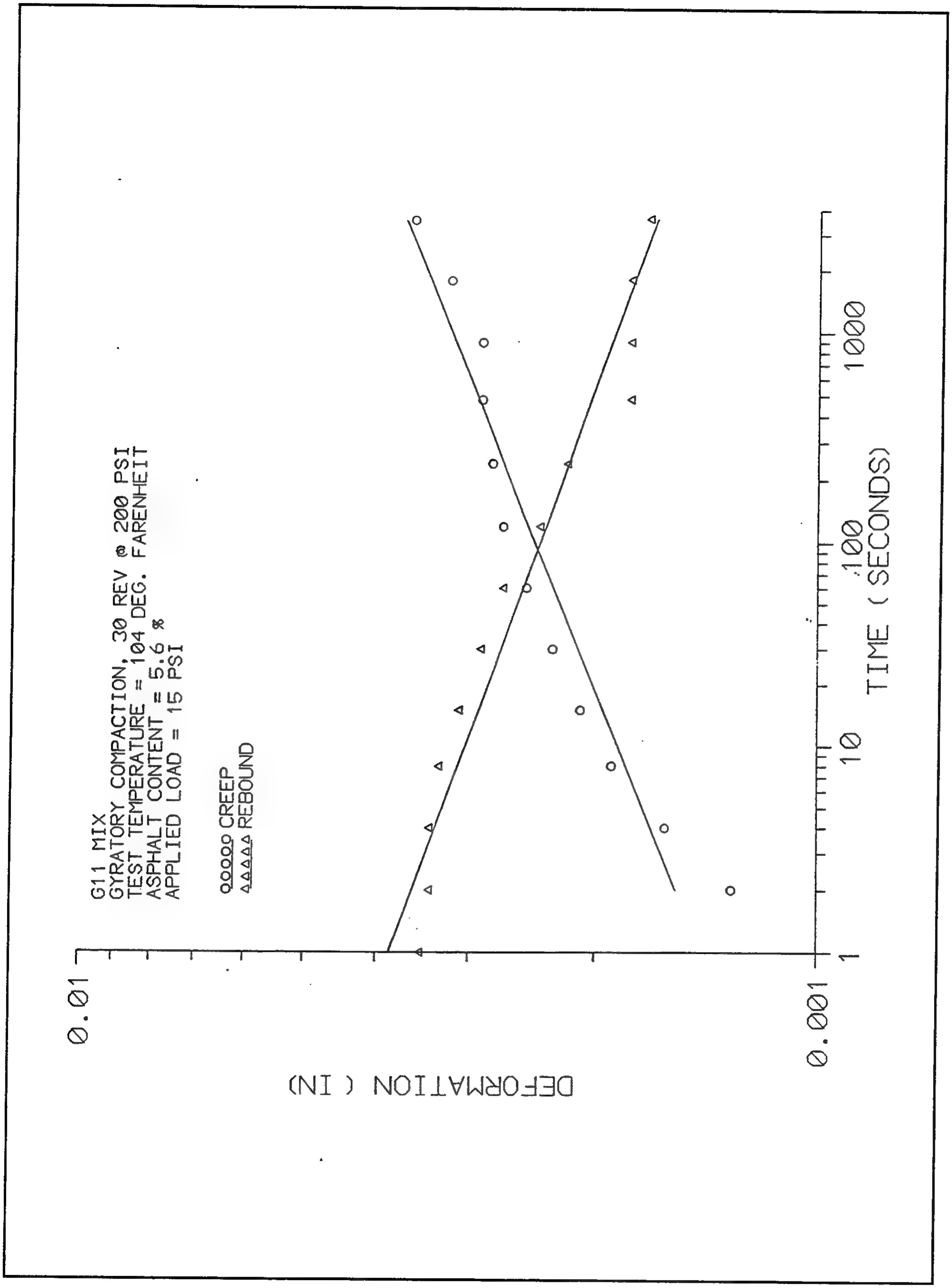

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
APPLIED LOAD = 15 PSI
OOOOO CREEP
ΔΔΔΔΔ REBOUND
0.01
0.001
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)

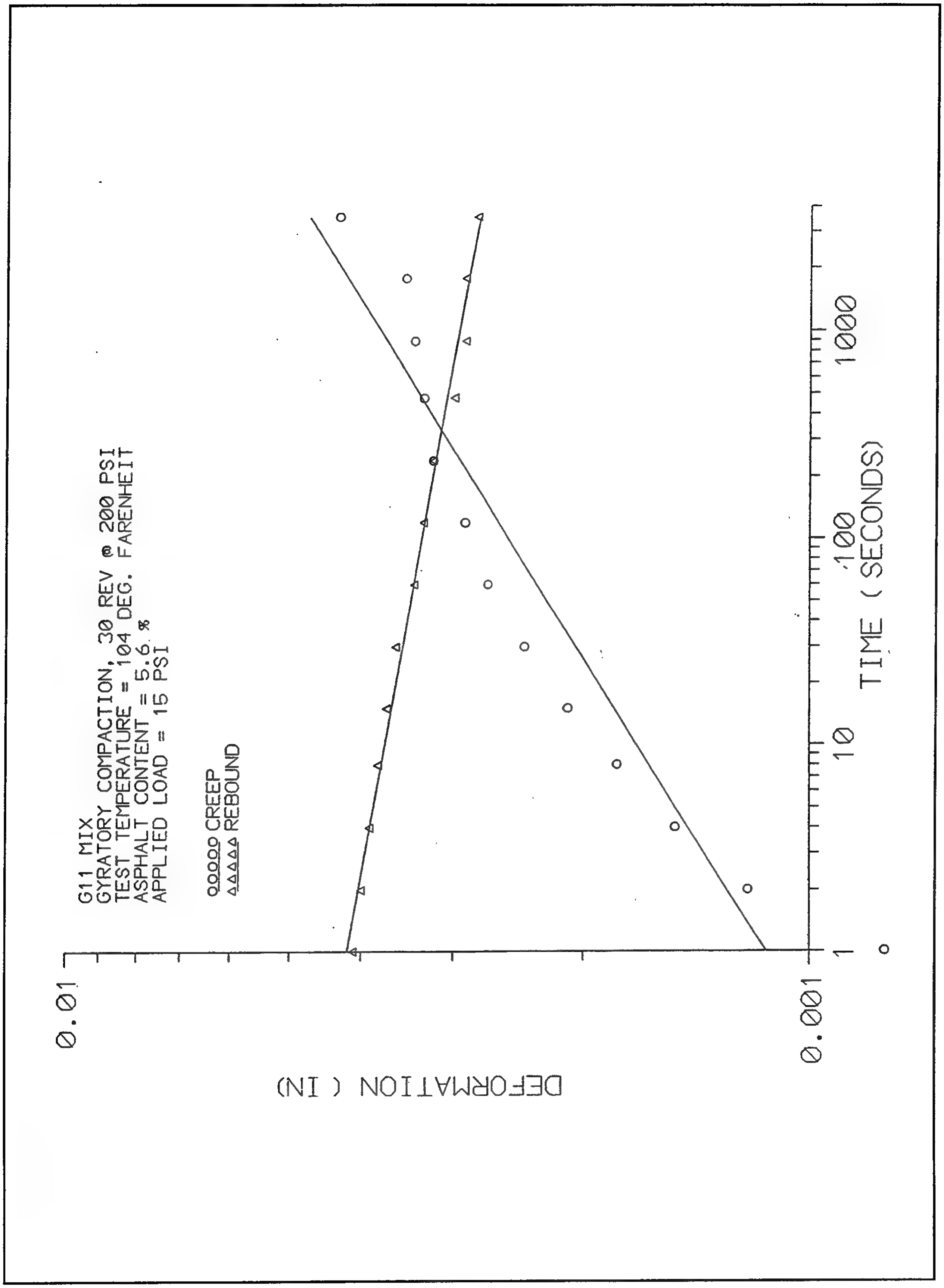

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
0.01
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)

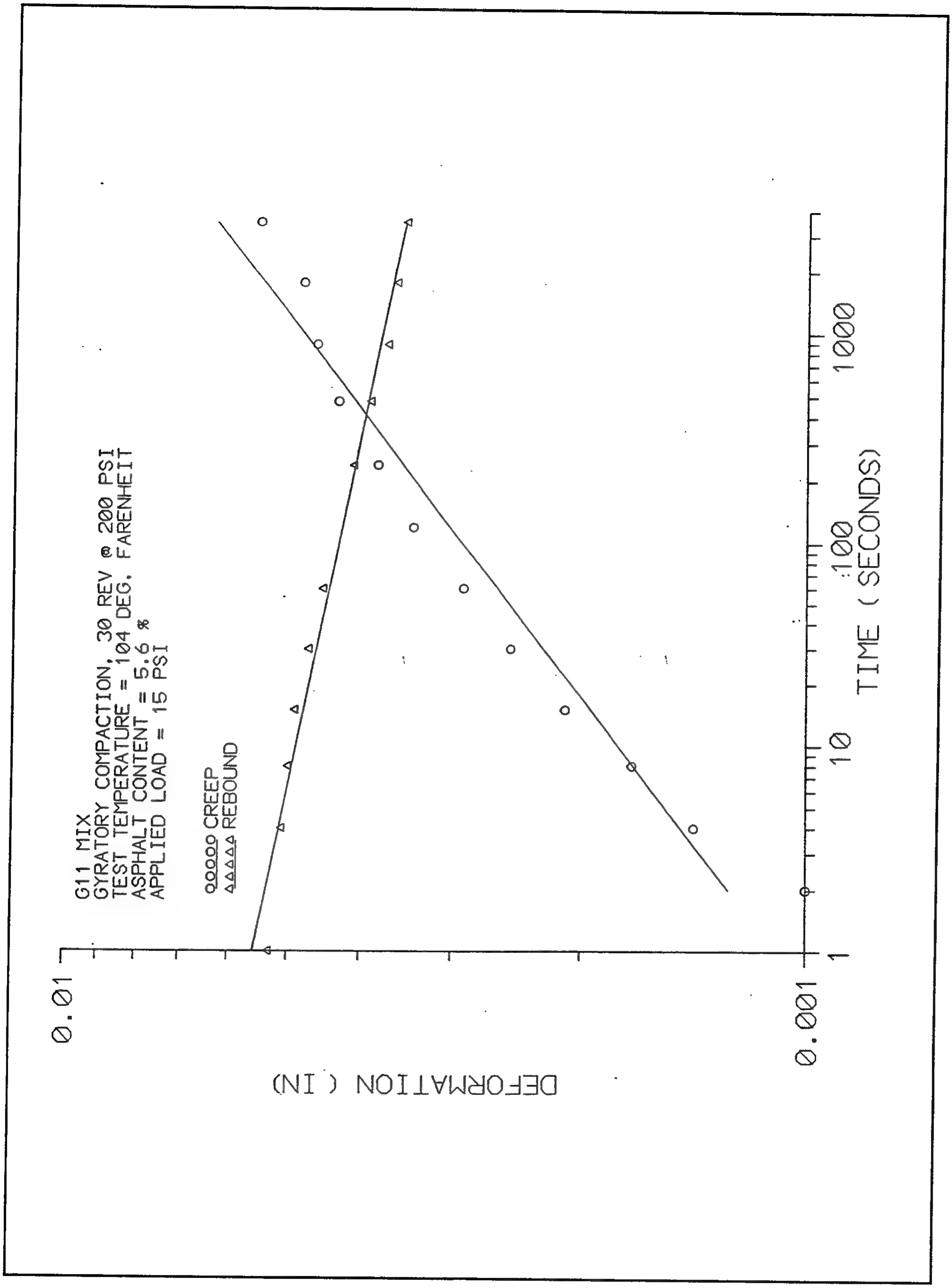

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
APPLIED LOAD = 40 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
0.01
0.001
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)

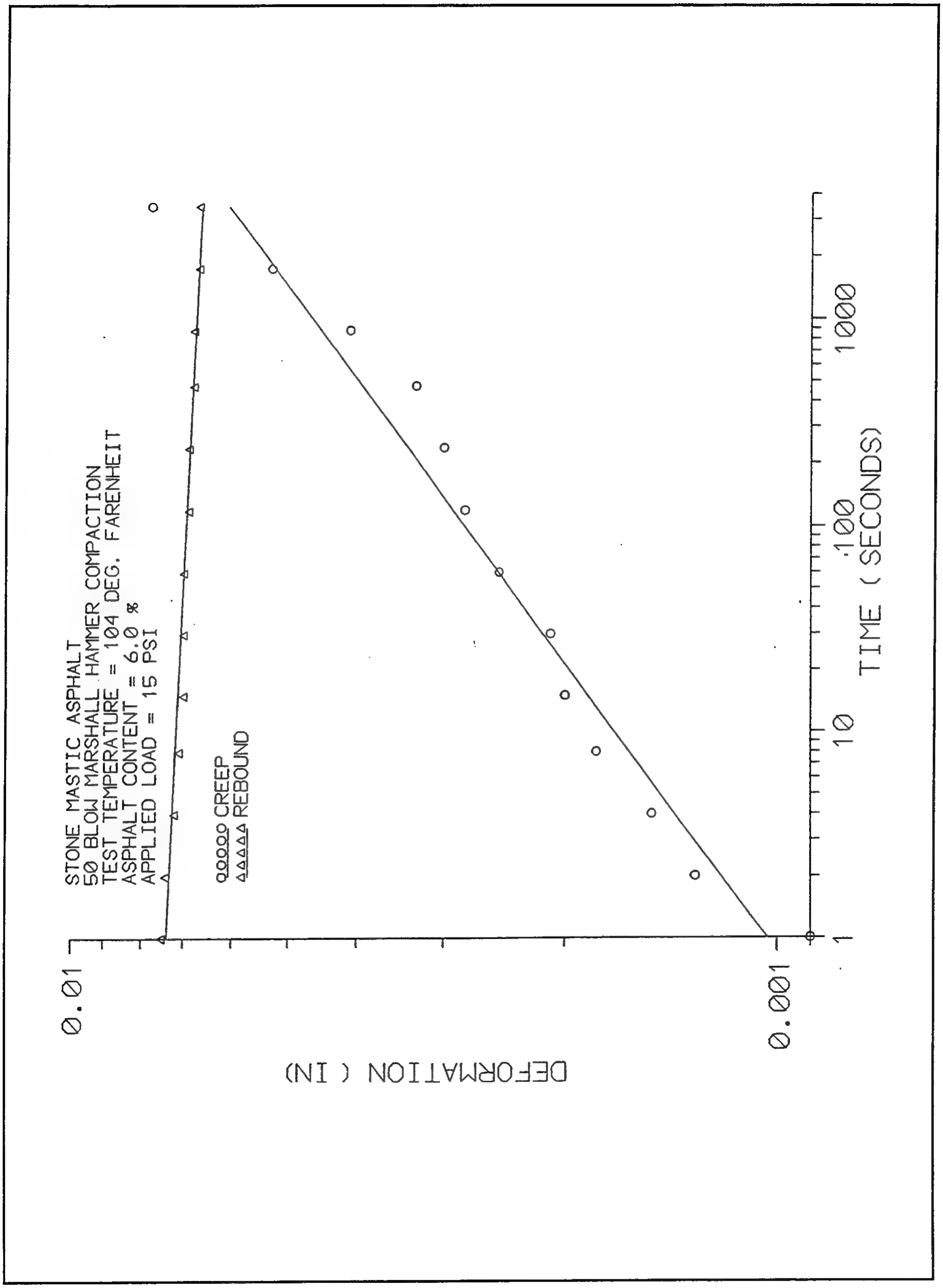


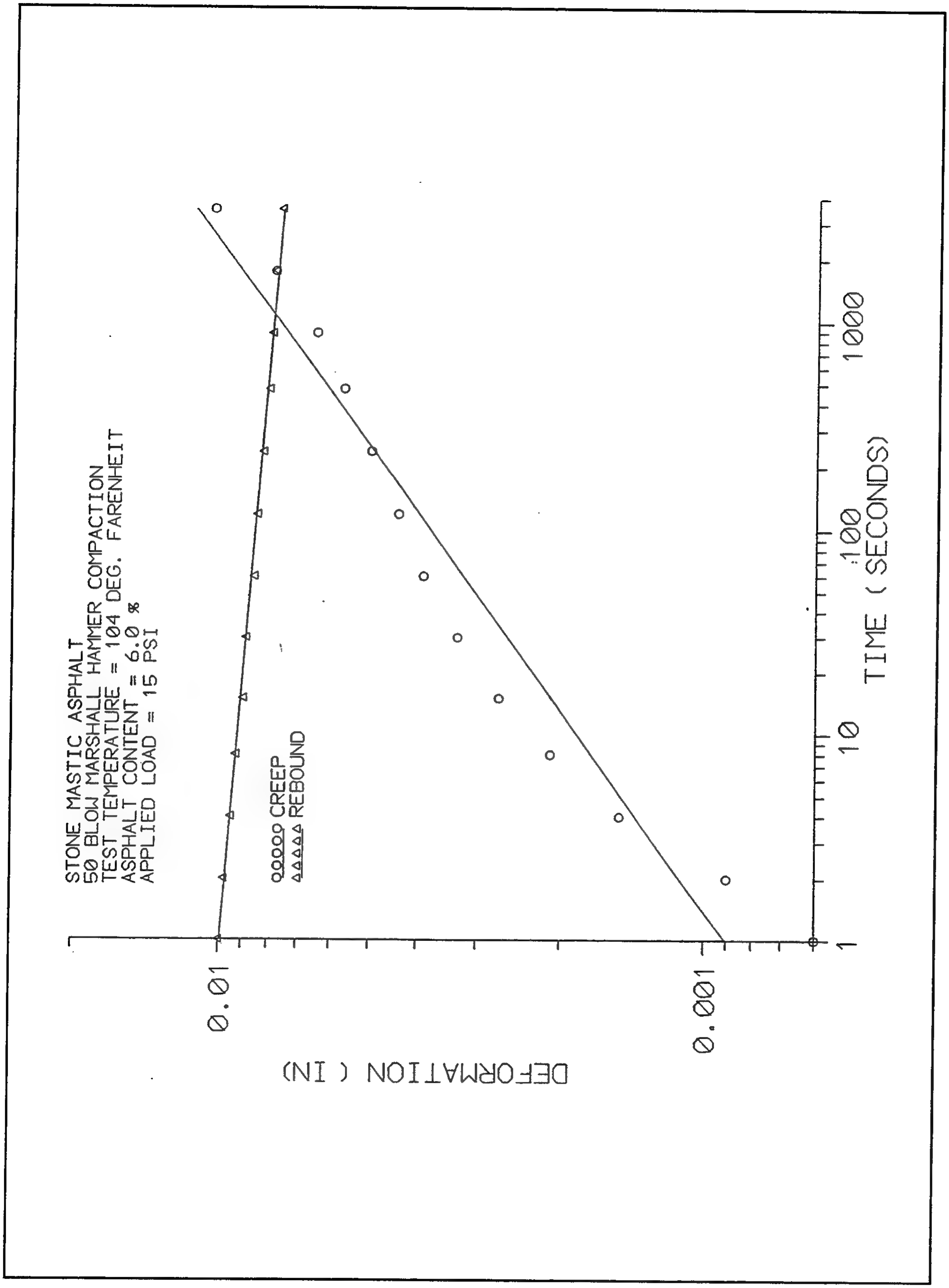
G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 77 DEG. FARENHEIT
ASPHALT CONTENT = 5.1 %
APPLIED LOAD = 40 PSI
ooooo CREEP
ooooo REBOUND
DEFORMATION (IN)
0.01
0.001
TIME (SECONDS)
10
100
1000

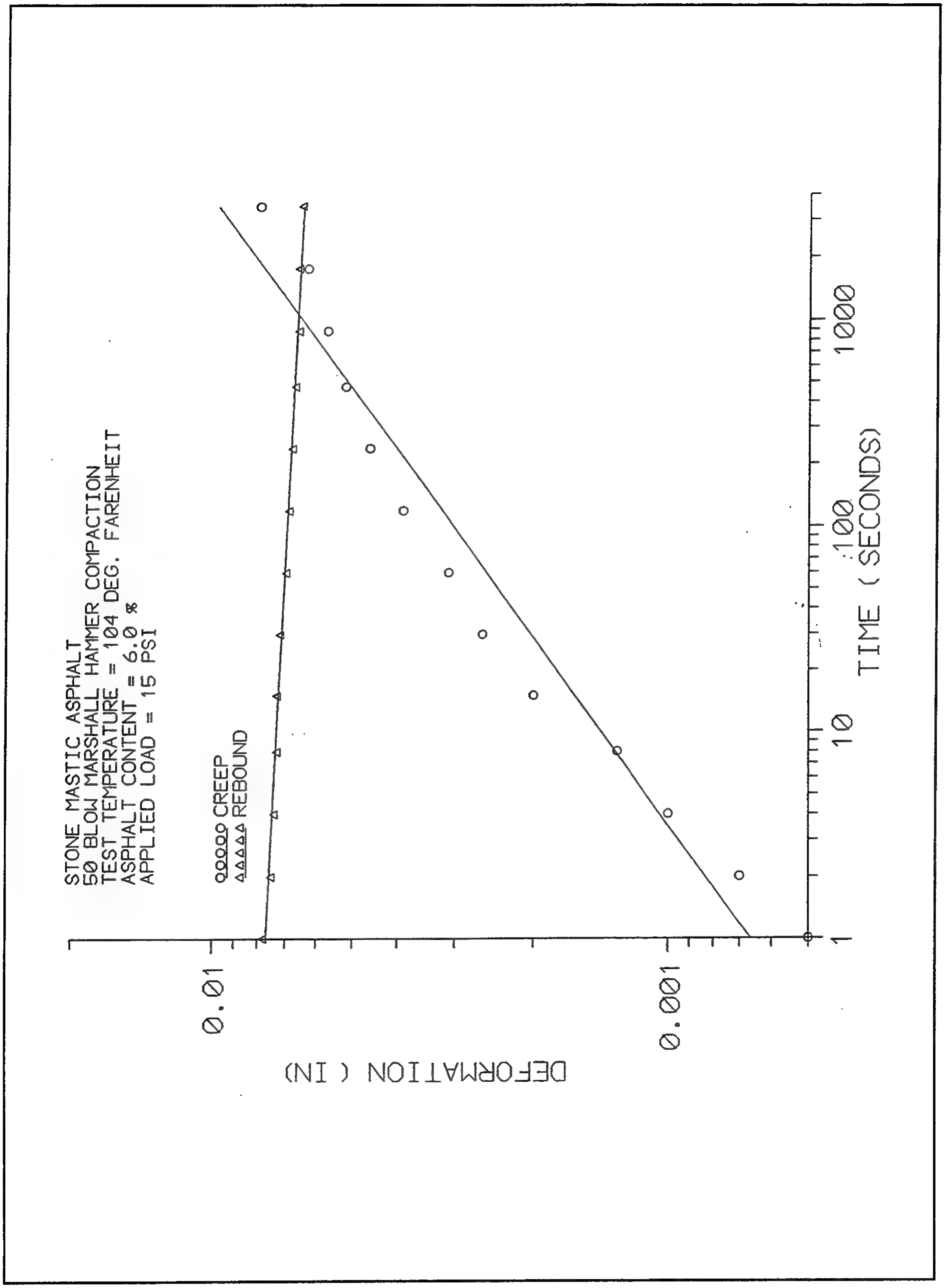

G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.6 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
0.01
0.001
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)


G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.6 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
0.01
0.001
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)


G11 MIX
GYRATORY COMPACTION, 30 REV @ 200 PSI
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 5.6 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
0.01
0.001
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)


STONE MASTIC ASPHALT
50 BLOW MARSHALL HAMMER COMPACTION
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
0.01
0.001
DEFORMATION (IN)
1
10
100
1000
TIME (SECONDS)


STONE MASTIC ASPHALT
50 BLOW MARSHALL HAMMER COMPACTION
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
DEFORMATION (IN)
0.01
0.001
TIME (SECONDS)
1
10
100
1000


STONE MASTIC ASPHALT
50 BLOW MARSHALL HAMMER COMPACTION
TEST TEMPERATURE = 104 DEG. FARENHEIT
ASPHALT CONTENT = 6.0 %
APPLIED LOAD = 15 PSI
ooooo CREEP
ΔΔΔΔΔ REBOUND
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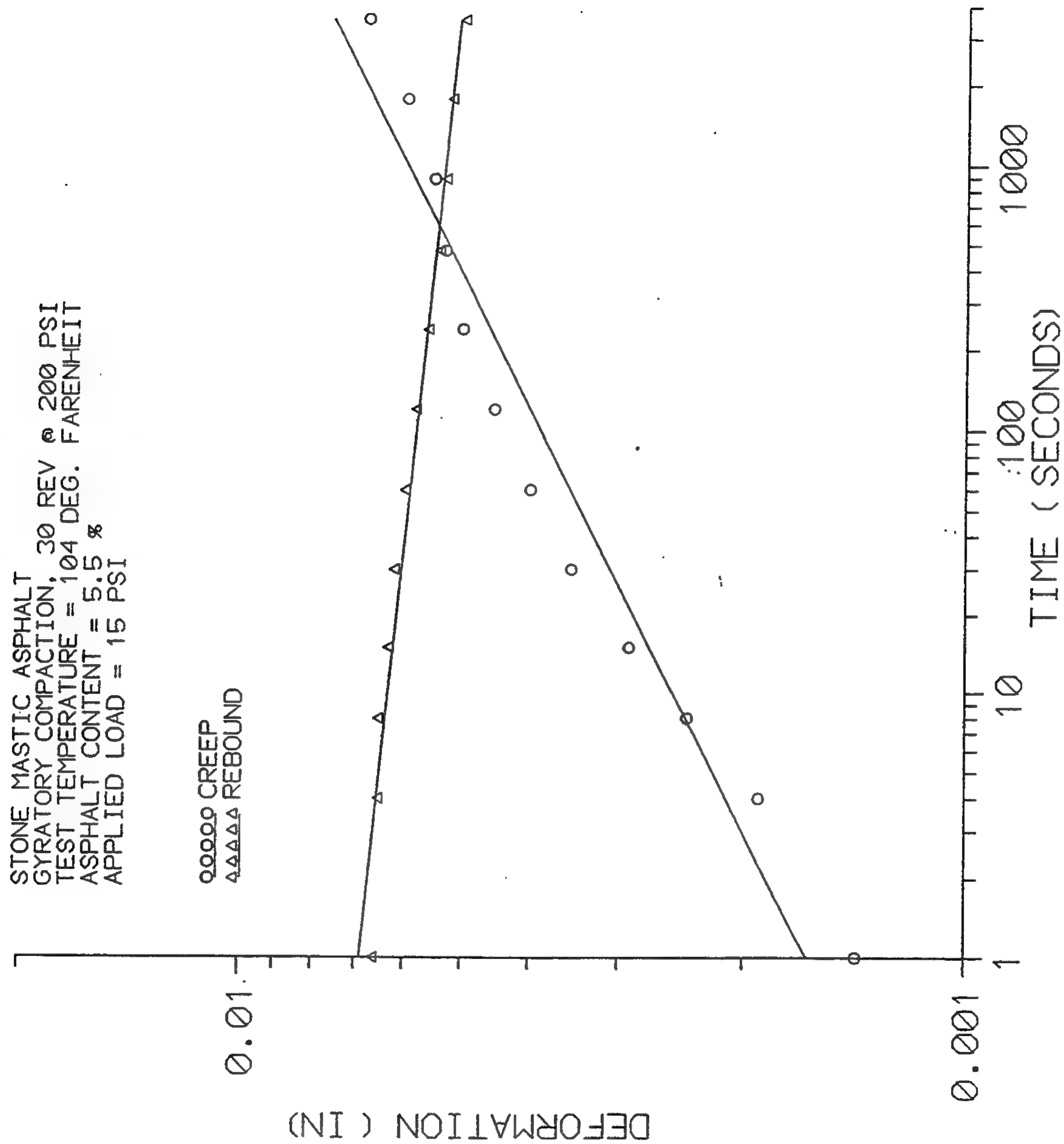

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APPLIED LOAD = 15 PSI
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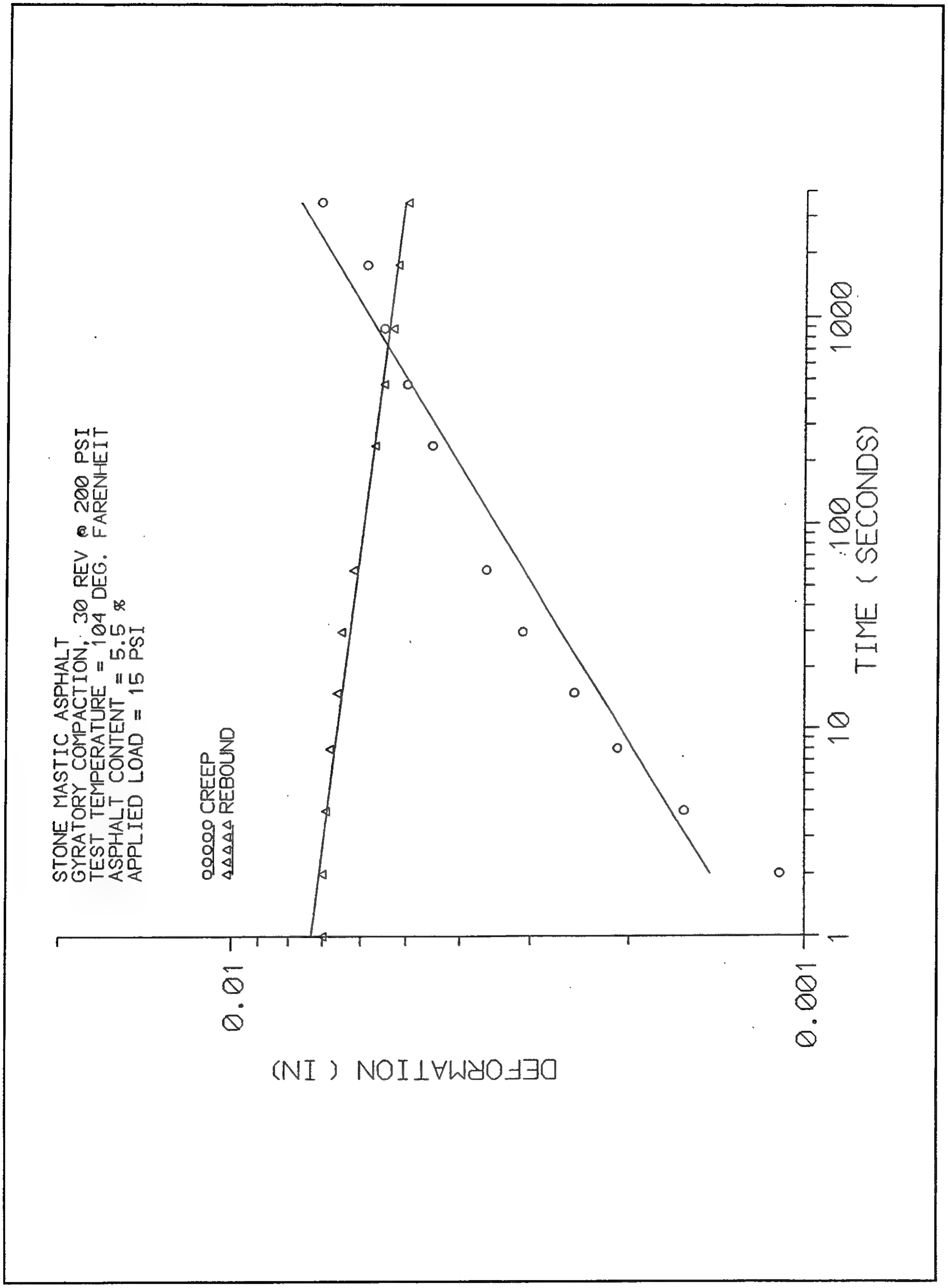
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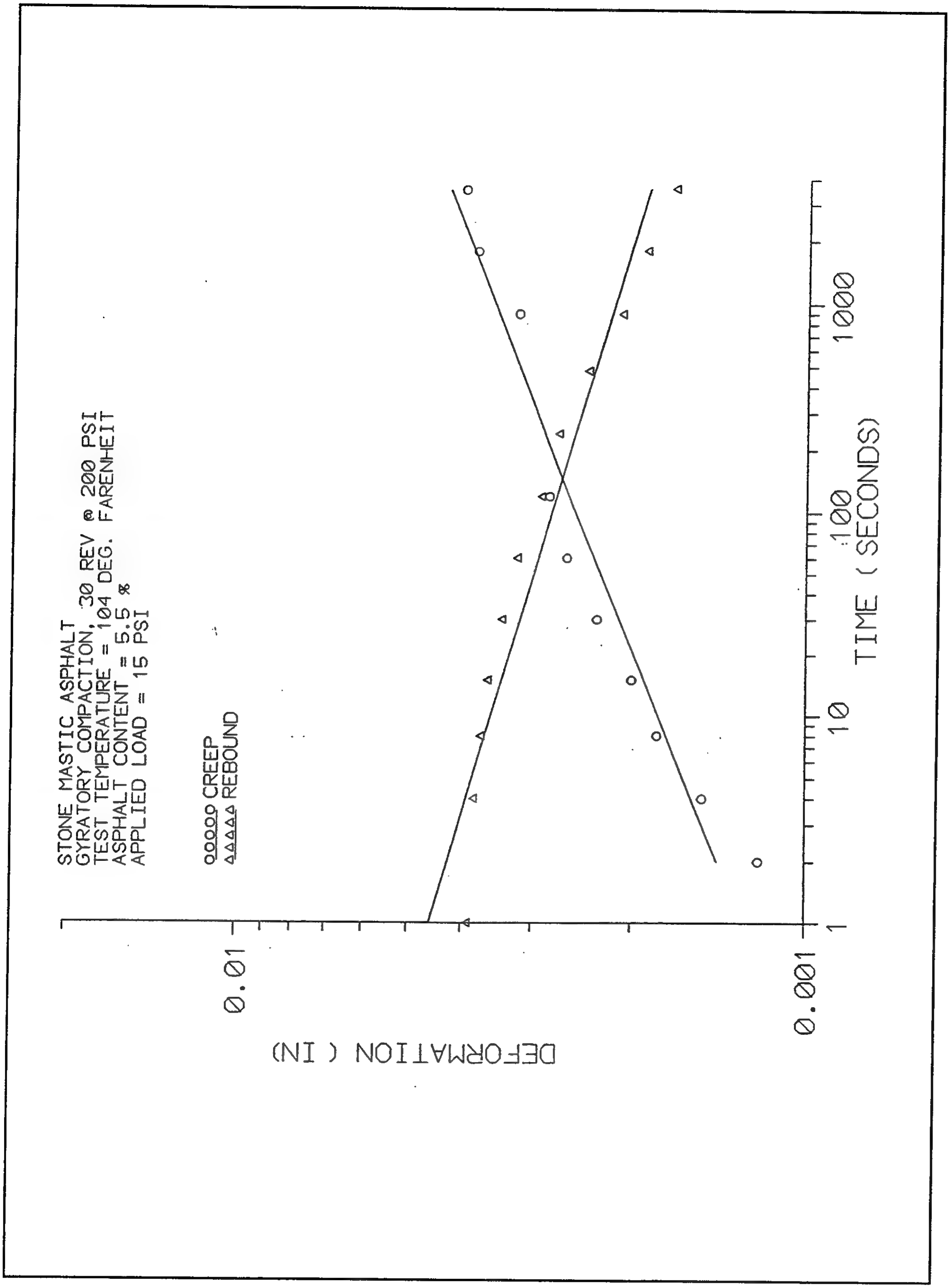
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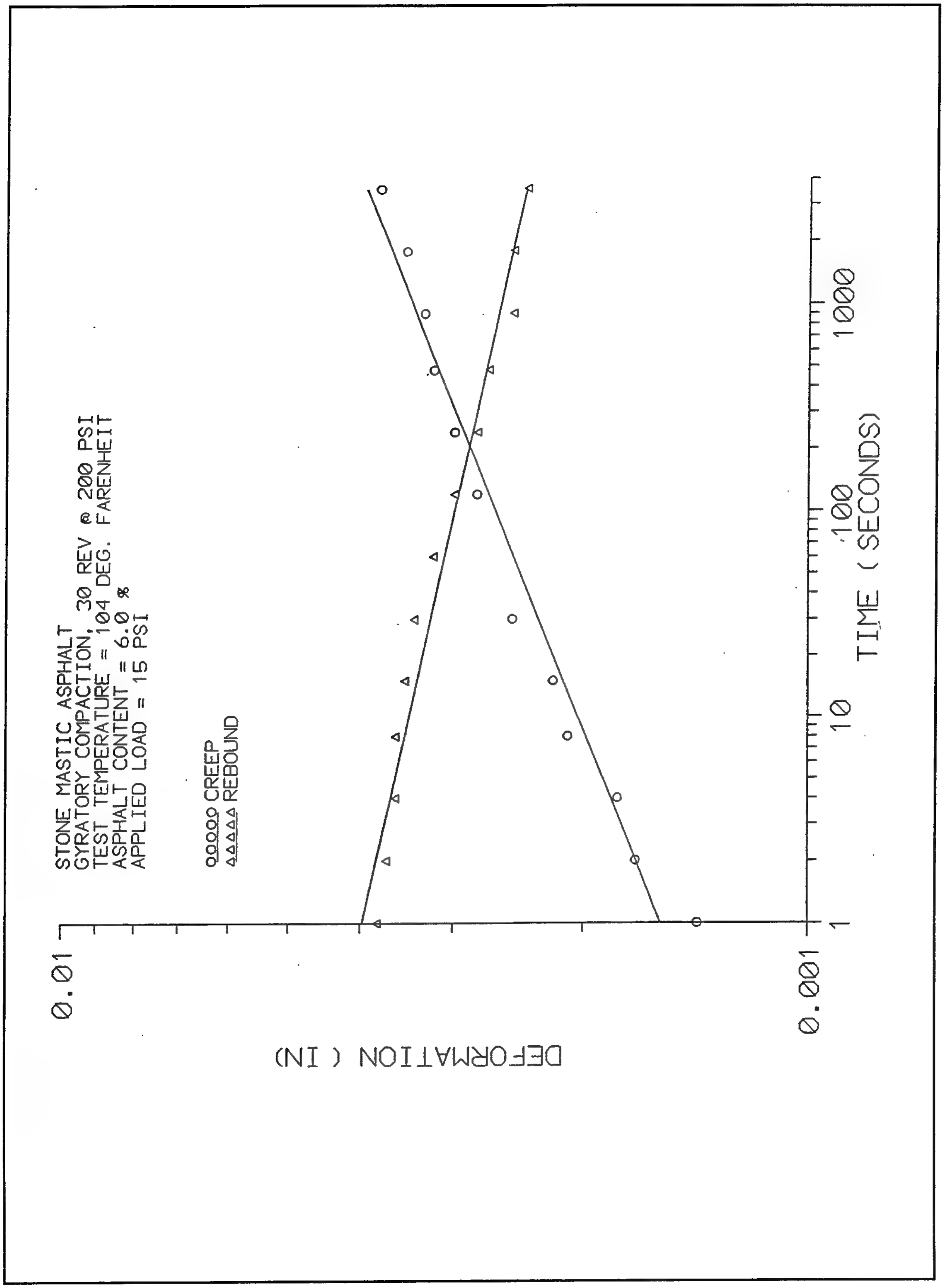
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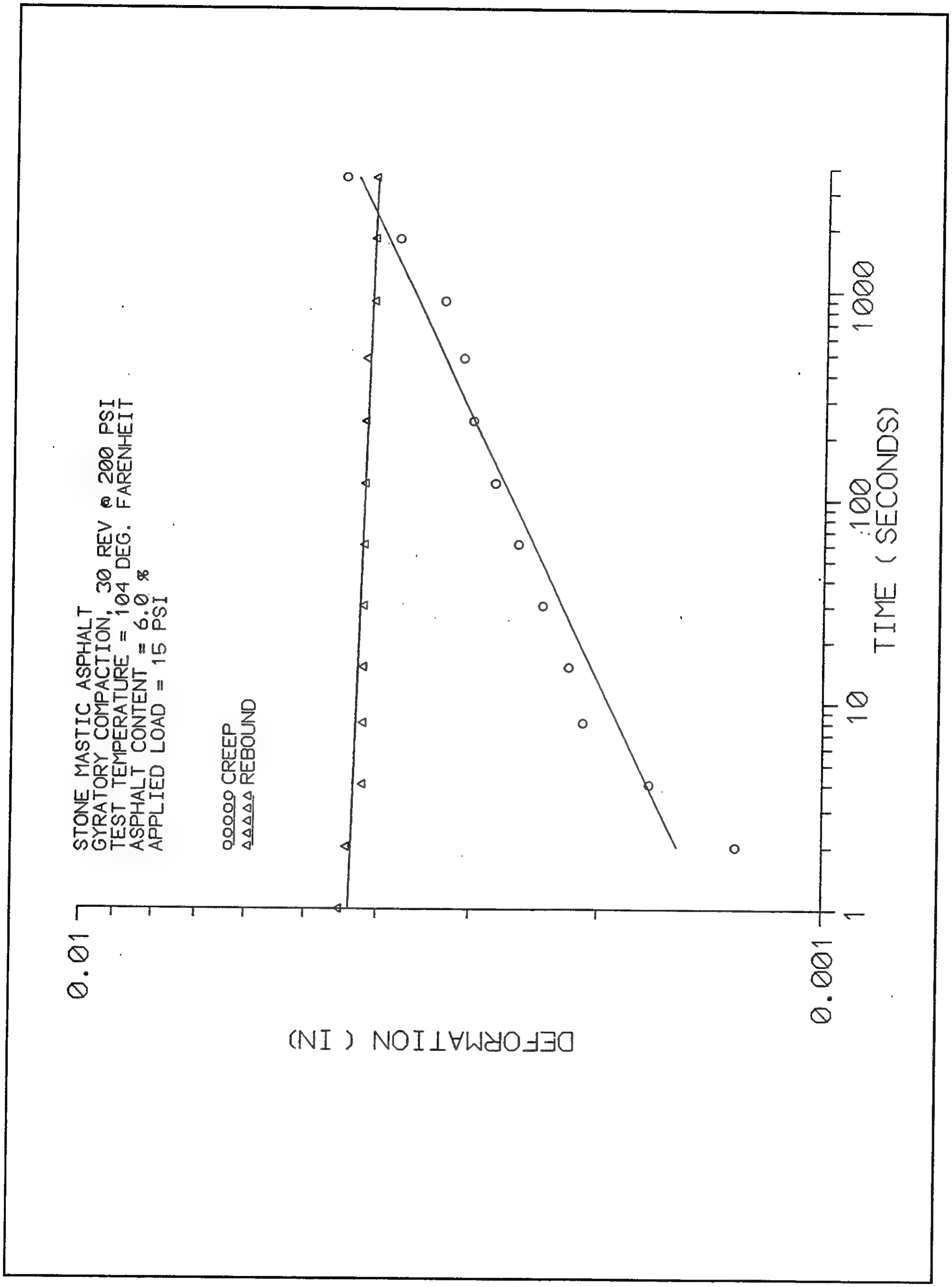
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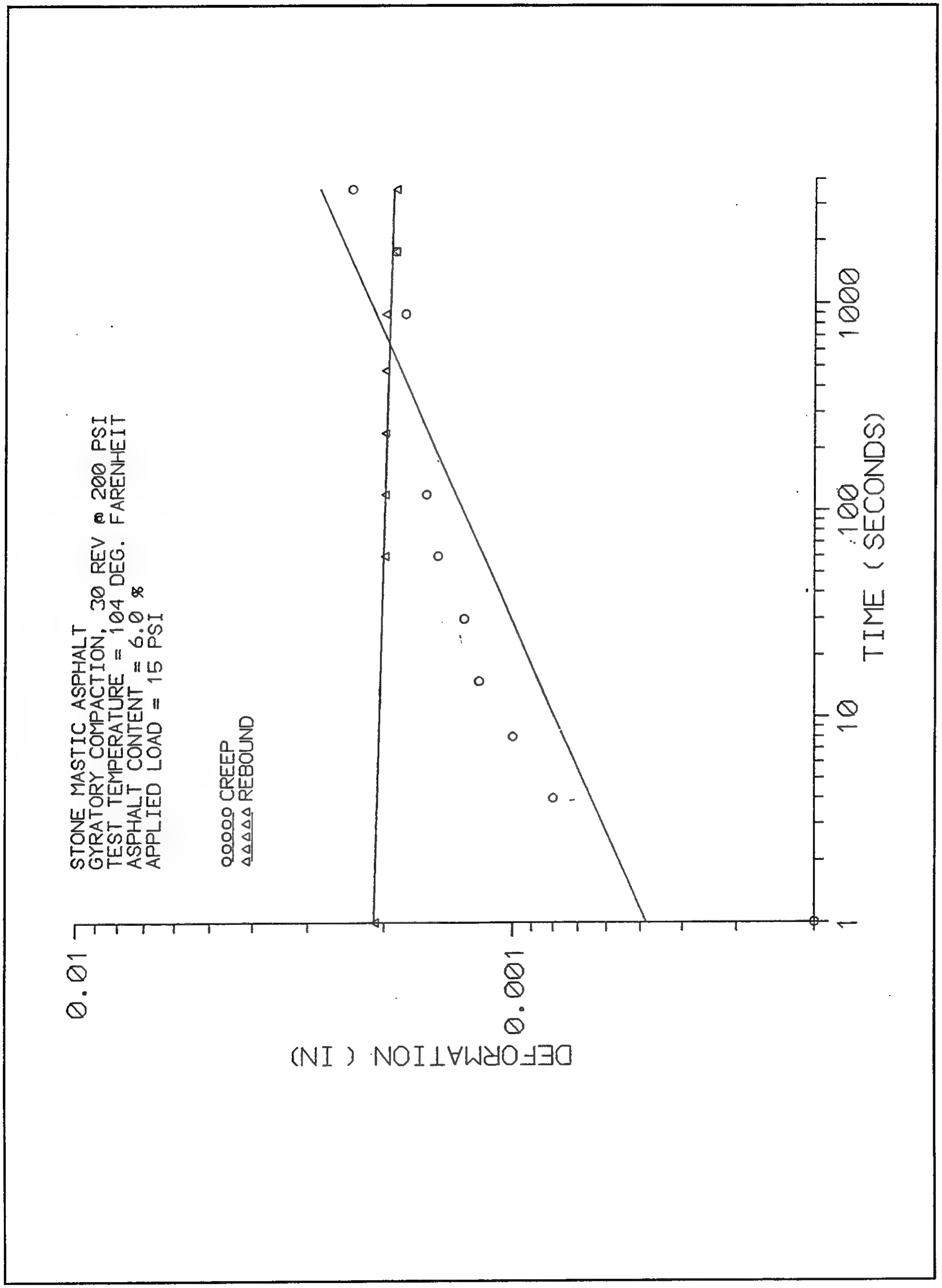
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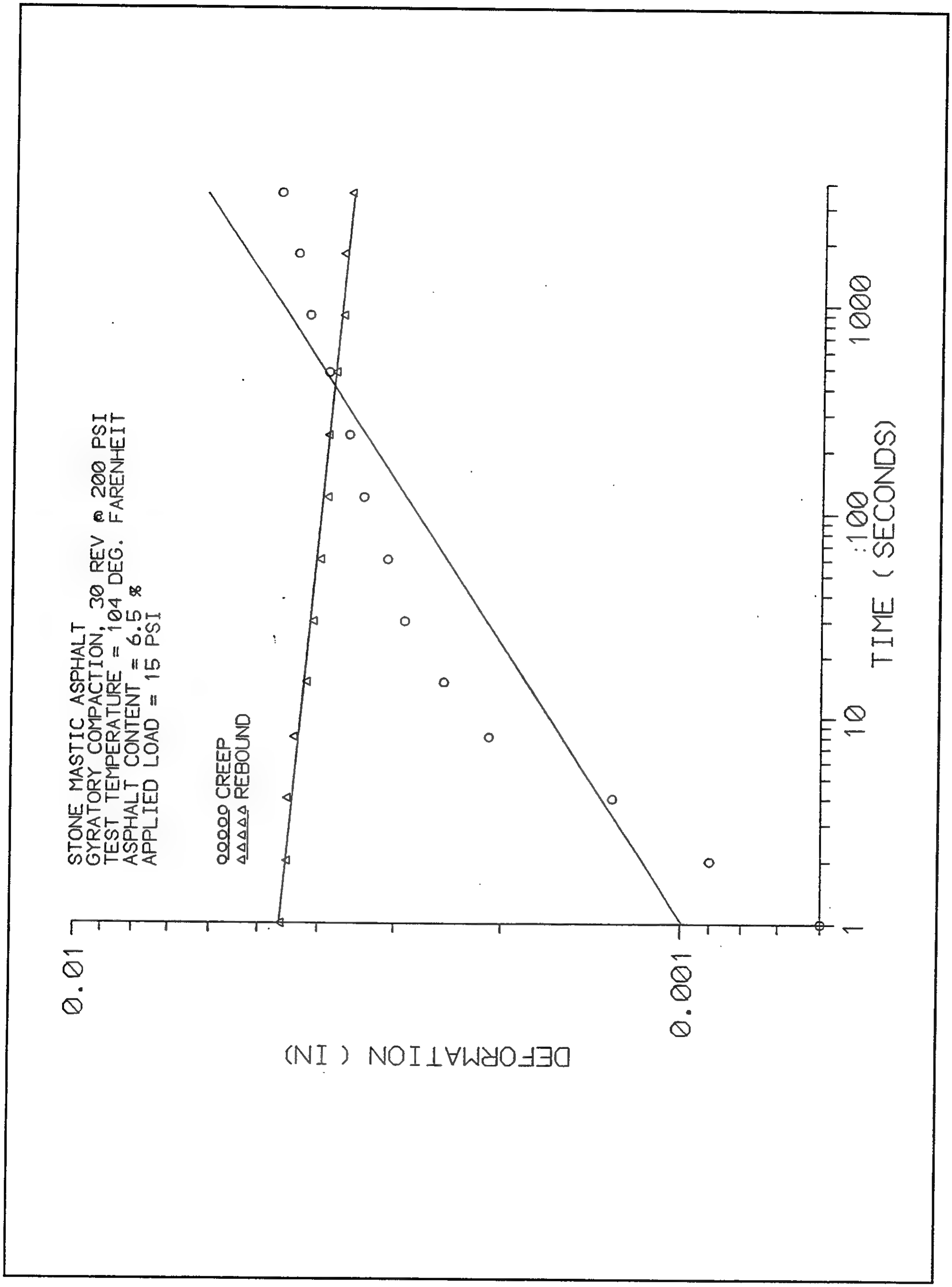
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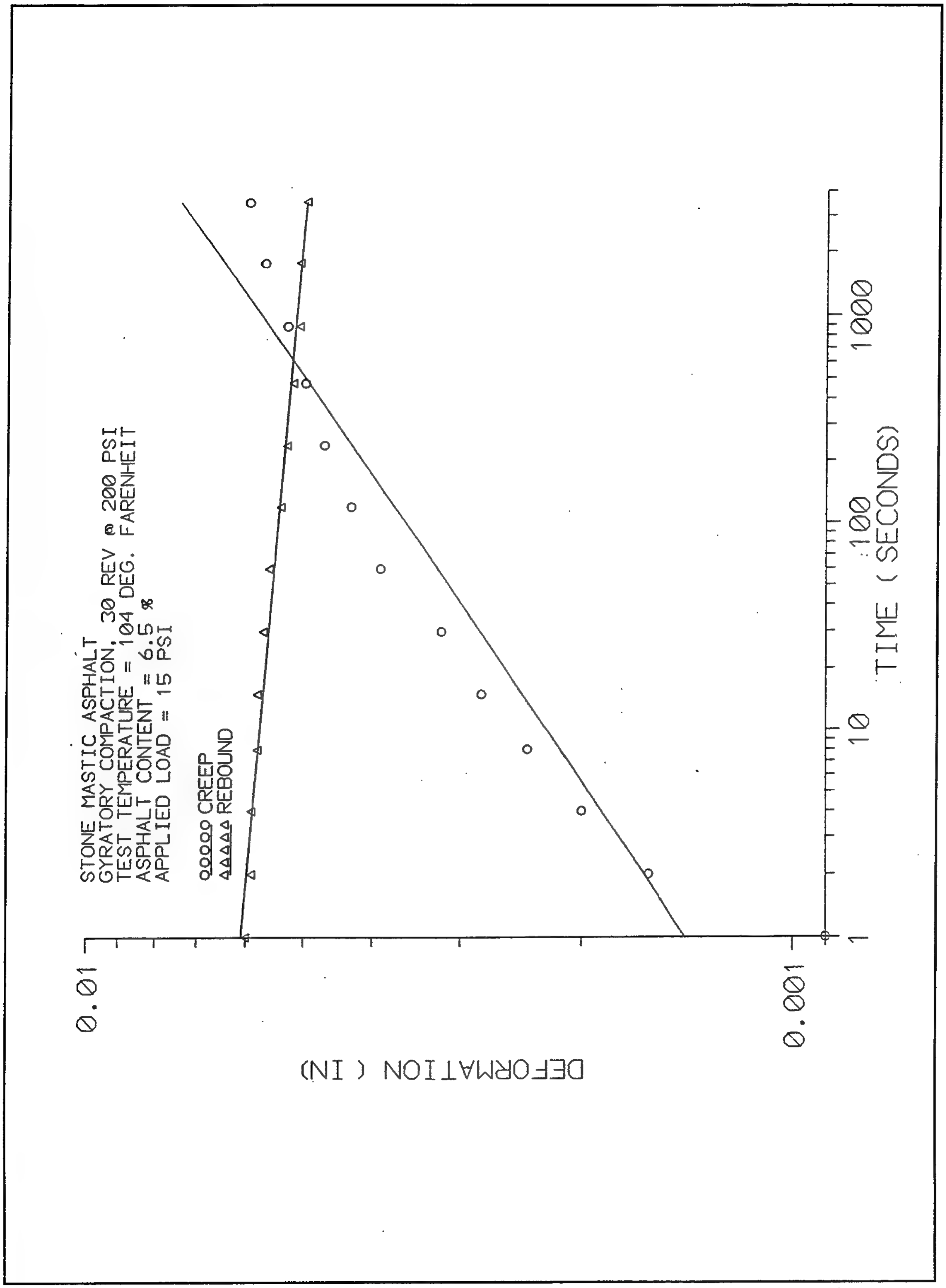
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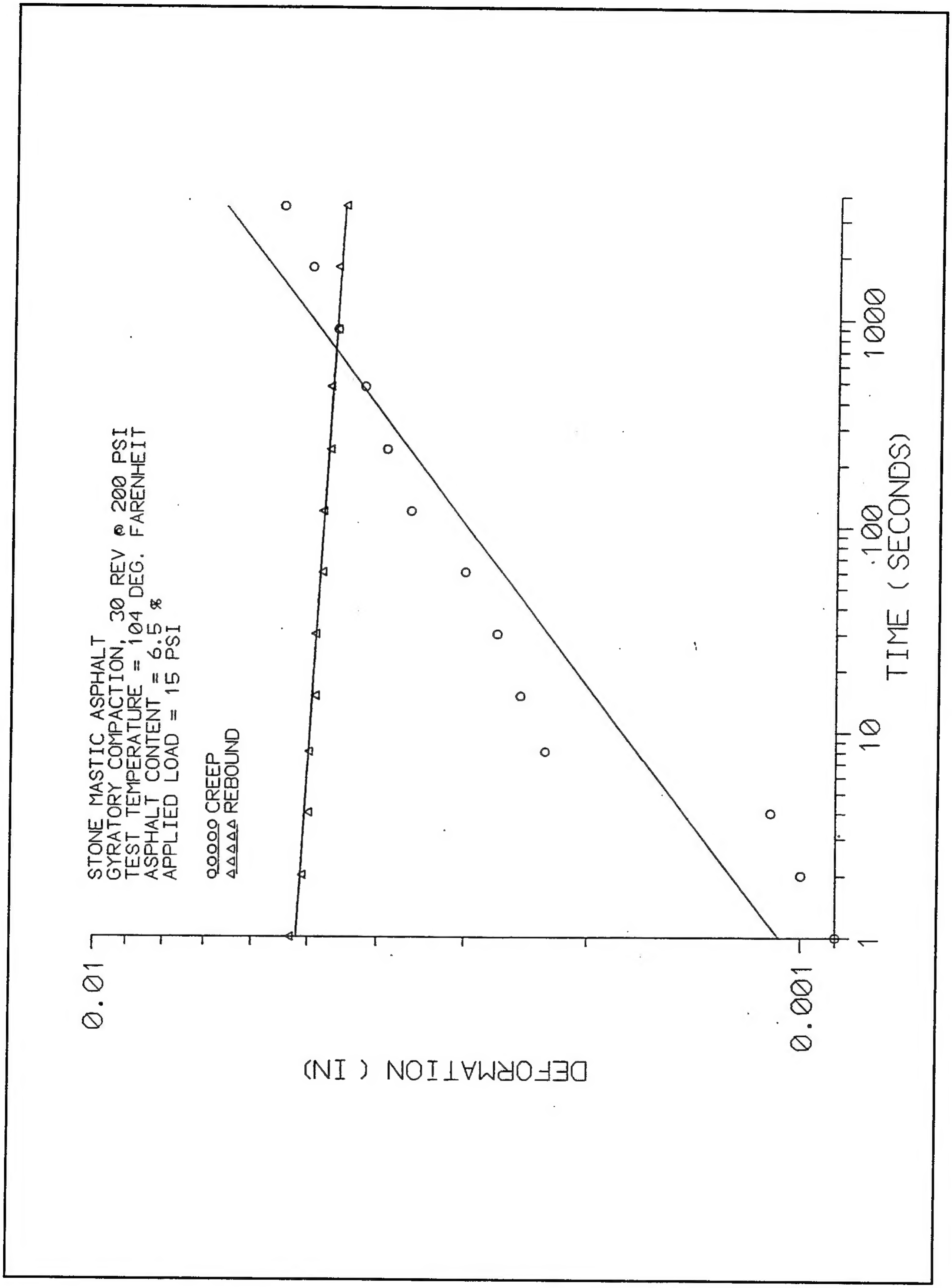
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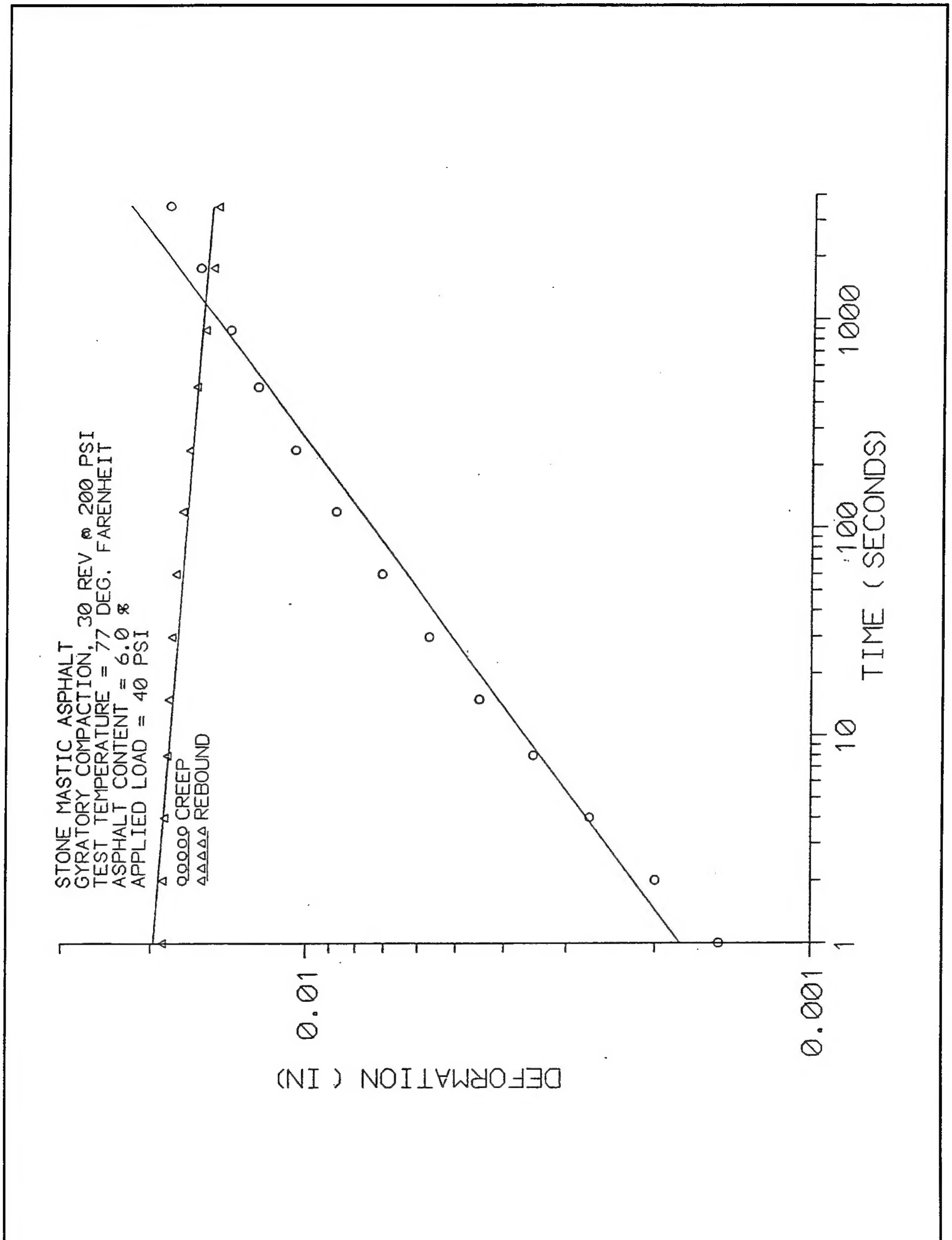
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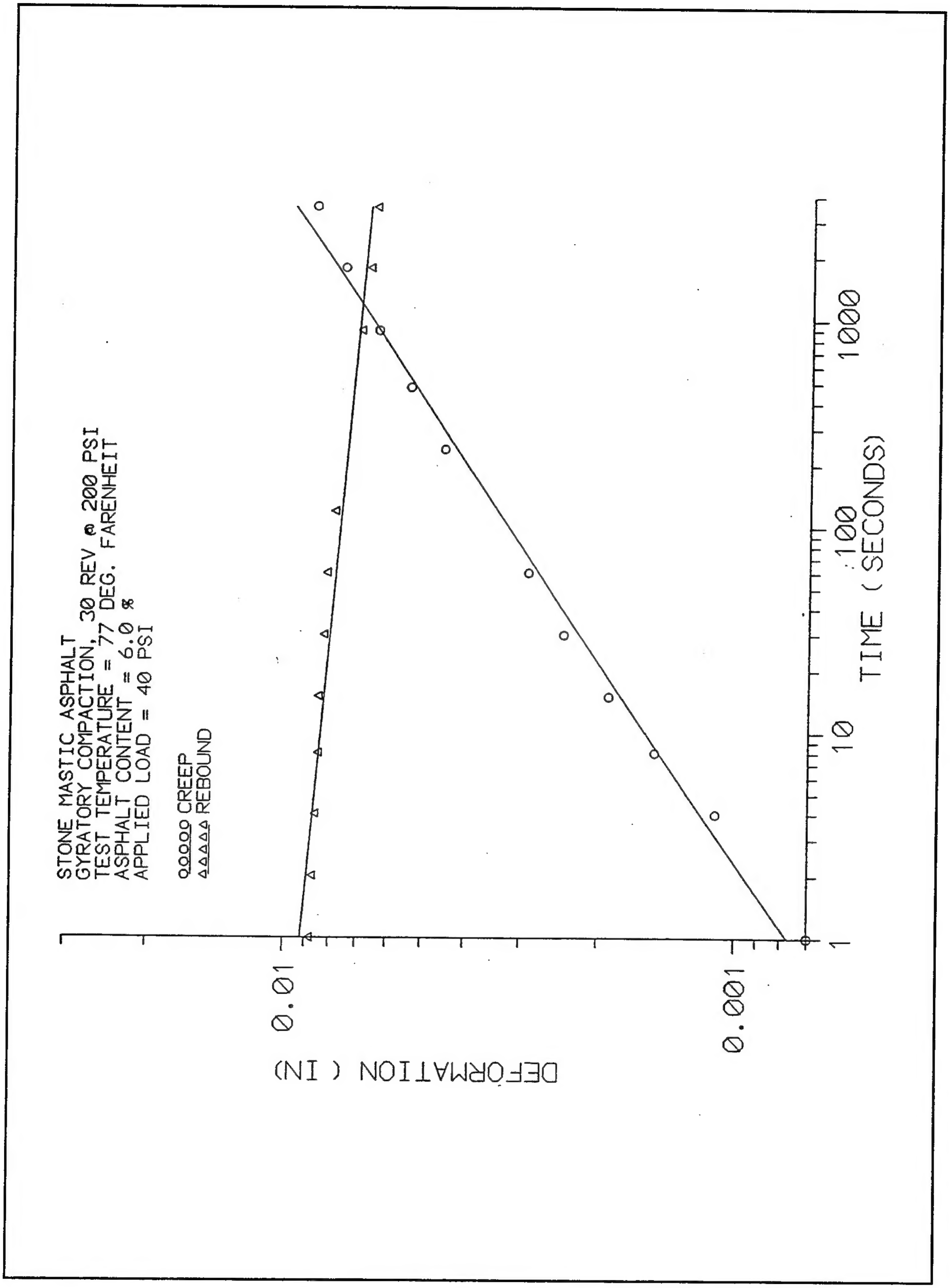
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13. ABSTRACT *(Maximum 200 words)*

This report documents a study that evaluated the mixture properties and field performance of stone matrix asphalt (SMA) mixtures. The study included a review of European practices and initial U.S. experiences with SMA design and construction. SMA mixtures were compared to standard dense-graded hot-mix asphalt mixtures using standard Marshall properties, indirect tensile tests, and unconfined creep-rebound tests.

SMA design and construction parameters were investigated during two construction projects. SMA was placed on a taxi way on the Royal Air Force Lakenheath, United Kingdom. A roadway and intersection were overlaid with SMA at Edwards Air Force Base (AFB), California. The performance of the SMA at Edwards AFB, after 3 years of use, is reported.

14. SUBJECT TERMS

Cellulose fibers	Marshall
Draindown test	Stone matrix asphalt
Hot-mix asphalt	Unconfined creep-rebound
Indirect tensile	

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